GEOTECHNICAL REPORT

SR 542/Squalicum Creek to Bellingham Bay - Fish Passage

03.37XL6093, SR 542, MP 03.37 - 03.52

Washington State Department of Transportation
Multimodal Development & Delivery
Construction Division
Geotechnical Office
1655 2nd Avenue SW
Tumwater, WA 98512-6951

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MICHAEL EUGENE SCHMITZ
STATE OF WASHINGTON
54223
REGISTERED PROFESSIONAL ENGINEER
2022.09.27
16:35:01-07'00'

State of Washington
1615
Licensed Geologist
Rolf B. Hyllseth
9/26/2022

Prepared by:	Mike Schmitz, P.E. Geotechnical Engineer Hart Crowser, a division of Haley & Aldrich		Rolf B. Hyllseth, P.E., L.G. Geologist/Geotechnical Engineer Hart Crowser, a division of Haley & Aldrich
Reviewed by:	D. Todd Mooney, P.E. Senior Foundation Engineer Geotechnical Office	Agency Approval Authority:	Andrew Fiske, P.E. State Geotechnical Engineer

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1 INTRODUCTION

This report was prepared by the Washington State Department of Transportation (WSDOT) Geotechnical Office (GO) for use by the Northwest Region Project Engineer's Office (PEO) and the Bridge and Structures Office (BSO). The purpose of this report is to support the design and preparation of plans, specifications, and estimate (PS&E) documents for the fish passage at the State Route (SR) 542 crossing of Squalicum Creek at Mile Post (MP) 3.37 - 3.52 (Project). The proposed project will replace the existing 205-foot-long, 6-foot by 6-foot concrete box culvert with a bridge to improve fish passage while providing a safe roadway for the traveling public.

This report presents the results of our geotechnical investigation and contains geotechnical recommendations for design and construction of one single-span bridge structure, Mechanically Stabilized Earth (MSE) bridge abutments composed of Low-Density Cellular Concrete (LDCC) reinforced with welded wire mesh, shallow foundations, geologic and stability review of an adjacent potential historical landslide, and embankment additions proposed for the project. When the PS&E review documentation is completed for this project, our office will provide a Summary of Geotechnical Conditions for inclusion in the Contract Provisions.

The analyses, conclusions, and recommendations in this report are based on five drilled borings conducted as part of this project, groundwater monitoring wells installed in three of the five borings, one Cone Penetrometer Test (CPT), laboratory testing on select samples collected from the borings, published geologic information of the site and vicinity, and our experience with similar geologic materials. The exploratory borings are assumed to be representative of the subsurface conditions throughout the project area. If, during construction, subsurface conditions differ from those described in the explorations, we should be advised immediately so we may reevaluate our recommendations and provide assistance.

2 PROJECT OVERVIEW

The project site is located on SR 542 in Whatcom County, approximately 0.6 miles northeast of Bellingham, Washington. The site location is shown on Figure 1. The existing project alignment crosses Squalicum Creek approximately between the State Route (SR) 542 Mile Posts (MP) 03.37 and 03.52, which is a two-laned roadway. Most of the current project alignment is on an historically placed embankment across the natural ravine.

The project includes construction of the following:

- One bridge structure, including:
    - MSE Wall Abutments composed of LDCC and welded wire reinforcement;
    - Shallow foundations supporting the bridge;

- Sheetpile walls around the base of the abutment; and
- Sheetpile wall for scour protection.

- Additional embankment fill for final grading.

Preliminary geologic review of the site location identified evidence of historical landsliding within the slope to the southeast of the existing embankment. Therefore, additional field reconnaissance, exploration, and analysis have been conducted to evaluate the risk of potential future landsliding impacting the proposed bridge structure.

Unless otherwise noted, the vertical datum used for the project is the North American Vertical Datum of 1988 (NAVD88), and the horizontal datum is the North American Datum of 1983, State Plane North (NAD83).

3 SITE INVESTIGATION

The project field exploration program consisted of drilling five test borings and one CPT along the alignment of the proposed bridge structure and potential landslide area. The program also consisted of installing and monitoring field instrumentation. Information obtained during the field exploration program was used to characterize the subsurface conditions at the proposed culvert.

3.1 SUBSURFACE EXPLORATION PROGRAM

Six geotechnical explorations were completed at the project site to characterize subsurface soils, collect samples for index and advanced laboratory testing, and install piezometers/inclinometers for groundwater and slope monitoring. These include five Standard Penetration Test (SPT) borings and one CPT. Locations of the test borings are shown on Figure 2. The five SPT test borings were drilled between June 16, 2020 and February 24, 2021. The CPT was performed on February 22, 2021.

The five test borings were drilled using a casing advancer. Standard penetration tests (SPTs) were performed in the test borings. Test borings were generally sampled in approximate 2.5-foot intervals or 5-foot intervals. One CPT was performed by ConeTec. Further details describing the field exploration methodologies are provided in Appendix A.

A WSDOT drill inspector collected soil samples and completed a visual classification of recovered samples. Following completion of drilling and sampling, the WSDOT drill crew conducted bail and recharge testing at each boring location. The logs and graphical legend of the test borings are included in Appendix A. The boring logs provide additional details on the sampling.

3.2 Laboratory Testing

Laboratory testing was performed by the WSDOT Materials Laboratory and by Hart Crowser, a division of Haley & Aldrich (Hart Crowser) of Seattle on selected soil samples for the purposes of classification and development of soil engineering properties, such as effective friction angle, over-consolidation ratio (OCR), undrained shear strength ration versus depth. Index tests performed included natural moisture content, Atterberg limits, grain size tests, and hydrometer analyses. Advanced tests performed included Constant Rate of Strain (CRS) consolidation tests, Direct Simple Shear tests (DSS), Triaxial Compression tests (isoCU), and Cyclic Direct Simple Shear tests (CDSS). To aid in selection of test samples, selected Shelby tubes were X-rayed by a subconsultant. Laboratory testing was performed in general accordance with appropriate ASTM International (ASTM) and American Association of State Highway and Transportation Officials (AASHTO) test methods. Laboratory test data is provided in Appendices B and C.

3.3 Field Instrumentation

Upon completion of the test boring H-2p-20, a 1-inch-diameter open standpipe piezometer was installed. Upon completion of test boring H-1vw-20 two vibrating wire piezometers (VWPs) were installed at depths of 30 feet (elevation 293 feet) and 120 feet (elevation 203 feet), respectively. Upon completion of test boring H-3vw-20, two VWPs were installed at depths of 20 feet (elevation 304 feet) and 120 feet (elevation 204 feet), respectively. The descriptions of the installed casing, piezometers and construction of the piezometers are included on the test boring logs and in Appendix A.

The piezometers have been continuously monitored since installation. Boring H-2p-20 was monitored using a Level Troll 500 data logger. Borings H-1vw-20 and H-3vw-20 were monitored using two VWPs as described in the previous paragraph. The results collected to date are presented in Appendix D. To optimize the benefit of this groundwater monitoring program, we will continue to monitor these piezometers until final construction bid documents are prepared and will incorporate the latest groundwater level readings in the Summary of Geotechnical Conditions at that time. All test borings with piezometers installed for this project will then be decommissioned in accordance with Washington Administrative Code (WAC) 173-160.

Upon completion of test borings H-4si-21 and H-5si-21, a 2.75-inch-diameter open standpipe was installed to allow installation of inclinometers to perform slope movement monitoring. These have been monitored since installation and the results collected to date are presented in Appendix E.

4 GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Site Geology

As part of this project, we reviewed available geologic data and provided site-specific geologic maps for the project alignment based on 1:100,000 scale 2010 geologic mapping available online from the Washington Department of Natural Resources (DNR; DNR 2010). The project site geology is depicted on Figure 3.

The project is underlain by Pleistocene-age Everson glaciomarine drift. Glaciomarine drift consist of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles, and occasional boulders. Additionally, large roadway embankment fills are located on the project site but are not shown on the geologic mapping.

In general, the conditions observed in the exploratory borings were consistent with the mapped geology of glaciomarine drift. Based on the site explorations, we interpret the contact between large roadway embankment fills and the top of glaciomarine drift to occur at approximately elevation 270 feet at the centerline of the existing culvert. This contact slopes upward toward both the east and the west along the historical ground surface contours of the pre-construction ravine.

4.2 Engineering Stratigraphic Units

The soils encountered in the test borings have been grouped into four Engineering Stratigraphic Units (ESUs) based on soil type, density, engineering properties, and geologic origin, as shown in Exhibit 4-1 and 4-2.

EXHIBIT 4-1: PROJECT-SPECIFIC ESUS

ESU #	Description	Elevation Range (feet)				
		Boring H-1vw-20	Boring H-2p-20	Boring H-3vw-20	Boring H-4si-21	Boring H-5si-21
1	Fill (Coarse Grained): Loose to Medium Dense Sand and Gravel with Silt	317-323	318-323	317-324	NA[1]	NA[1]
2	Embankment Fill (Fine Grained): Soft to Medium Stiff Clay and Silt with Sand	284-317	272-318	302-317	NA[1]	NA[1]
3	Bellingham Drift (Fine Grained): Medium Stiff to Very Stiff Clay and Silt with Sand (Bellingham Drift)	220-284	220-272	220-302	269-310	258-289
4	Bellingham Drift (Coarse Grained): Very Dense Sand and Gravel with Silt	193 -220	149-220	193-220	NA[1]	NA[1]

NOTES:

1. Did not encounter ESUs 1 and 4 in boring H-4si-21; did not encounter ESUs 1, 2, and 4 in boring H-5si-21.

EXHIBIT 4-2: PROJECT-SPECIFIC ESUS AND GENERAL DESIGN PARAMETERS AT BRIDGE ABUTMENT LOCATIONS

ESU #	Description	Design $(N_1)_{60}$ (bpf[1])	Total Unit Weight (pcf[2])	Effective Friction Angle (degrees)
1	Fill (Coarse Grained): Loose to Medium Dense Sand and Gravel with Silt	35	130	35
2	Embankment Fill (Fine Grained): Soft to Medium Stiff Clay and Silt with Sand	6	130	33
3	Bellingham Drift (Fine Grained): Medium Stiff to Very Stiff Clay and Silt with Sand (Bellingham Drift)	12	130	33
4	Bellingham Drift (Coarse Grained): Very Dense Sand and Gravel with Silt	81	135	38

NOTES:

1. bpf = blows per foot
2. pcf = pounds per cubic foot

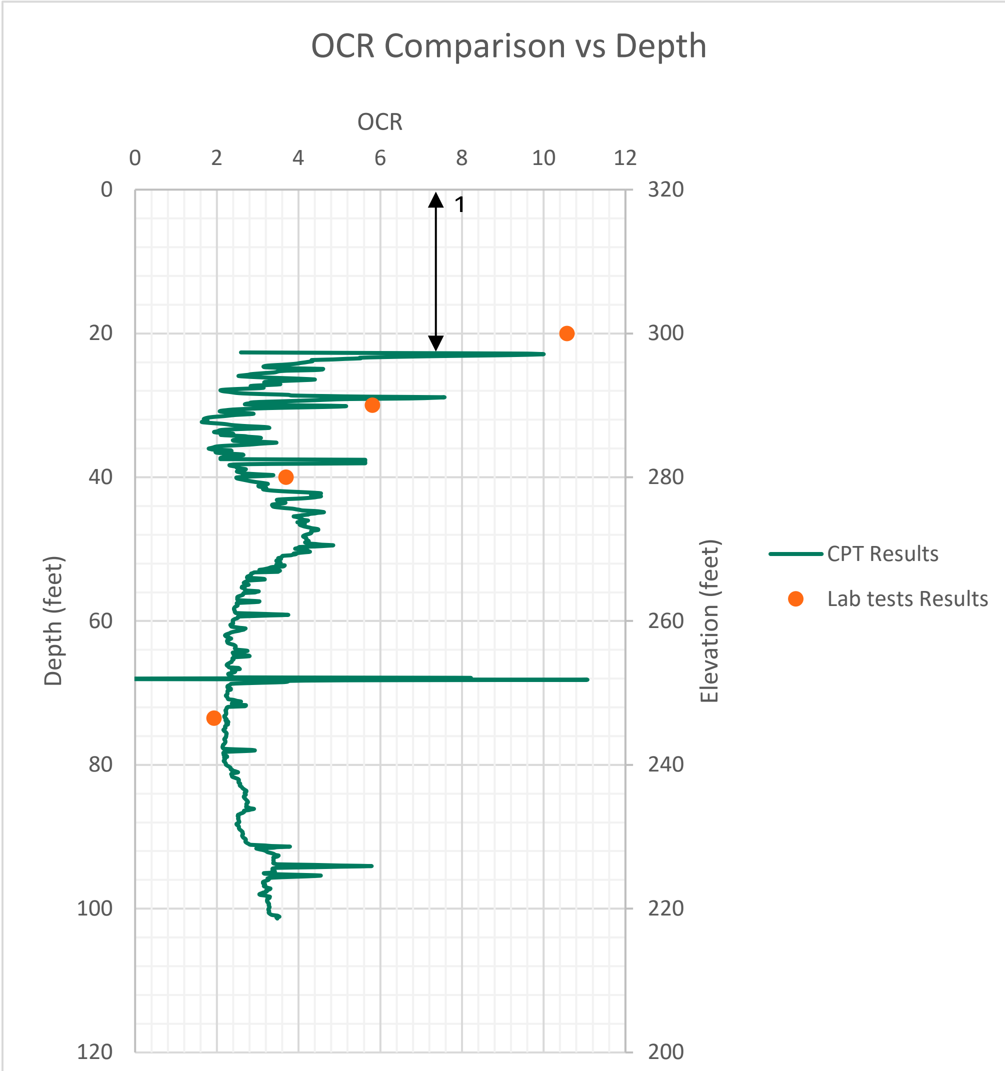
Figures 4 and 5 graphically depict our interpreted subsurface conditions along the project alignment, including the location and thickness of each ESU.

4.2.1 *Advanced Laboratory Test Results and Interpretation*

To develop advanced design parameters for each ESU, we performed four constant rate of CRS, three DSSs and two CDSSs on selected undisturbed samples collected from our subsurface explorations. We also performed two isoCUs tests on selected undisturbed samples. We selected the test specimens to obtain results across the cohesive ESUs 2 and 3 encountered at the site and at various depths. The intent of the testing was to validate the results of the CPT exploration performed through the existing embankment fill and the underlying fine-grained Bellingham Drift deposit (ESUs 2 and 3). Through this validation, we could determine site-specific undrained strength ratios for both the static and seismic design, as well as post-cyclic degraded strength ratios (see *Section 5.3.2*). We ran the tests at existing OCRs for each of the samples to account for the estimated stress profile at the site. Additional details regarding the laboratory procedures and results can be found in Appendix C.

To determine the stress history profile and design OCR value, we ran CRS testing on four selected soil samples. The results of the CRS testing suggested an OCR of approximately 10 at a depth of about 10 feet below ground surface (bgs) (Elevation 300 feet), 6 at a depth of about 20 feet bgs (Elevation 290 feet), 4 at a depth of 30 feet (Elevation 280 feet), and 2 at a depth of 77 feet (Elevation 245 feet). These values generally matched the OCR suggested from the CPT analysis in the *Interpretation of cone penetration Tests - a unified approach* (Robertson 2009). Exhibit 4-3, below, presents a graphical comparison between the OCR profile estimated from the CPT compared to the results of our laboratory testing.

EXHIBIT 4-3: OCR COMPARISON - CPT AND LABORATORY TESTING VERSUS DEPTH



NOTES:

1. CPT was pre-drilled to a depth of 22 feet bgs.

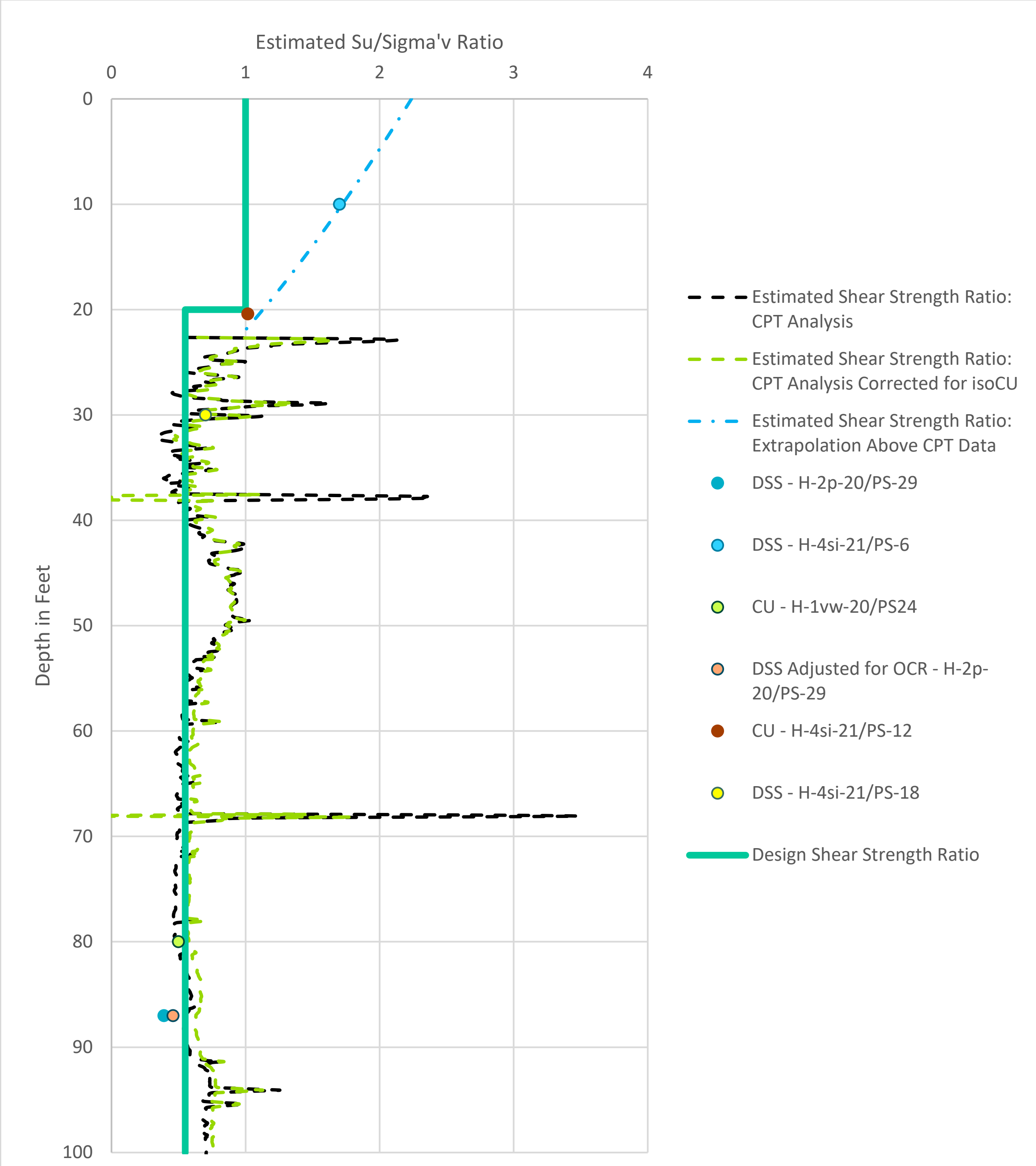
We next used the results of the DSS and isoCU triaxial tests to validate the undrained shear strength ratio estimated by the CPT. The undrained strength ratios for the DSS and isoCU results were calculated by normalizing the mobilized shear strength at failure (defined as the maximum shear stress achieved prior to 15 percent strain) by the in-situ effective confining pressure at the sample depth. We compared these values against the CPT data using two analyses methods:

- Interpretation of undrained shear strength ratio from CPTs, as displayed in the *Interpretation of cone penetration Tests - a unified approach* (Robertson 2009) using the CPT raw data; and
- Calculating the ratio based on the CPT OCR profile combined with the results of the isoCU triaxial tests. The conditions between and below explorations may be different.

The second method requires calculating the normally consolidated shear strength ratio using the drained friction angle estimated from the isoCU triaxial tests, according to equation 4-34 in the Electric Power Research Institute (EPRI) Soil Manual. We used this normally consolidated shear strength ratio as an input to the Stress History and Normalized Soil Engineering Parameters (SHANSEP) approach to address different OCRs, according to equation 4-37 of Electric Power Research Institute (EPRI) Soil Manual.

Exhibit 4-4, below, presents the graphical results of this CPT versus laboratory testing comparison, plotted against depth.

EXHIBIT 4-4: UNDRAINED SHEAR STRENGTH RATIO VERSUS DEPTH - CPT AND LABORATORY TESTING



Based on these results, we recommend using a design undrained shear strength ratio of 1.0 in the upper 20 feet in ESU 3 (between 300 to 320 feet at the top of the slope) and 0.55 below 20 feet in ESU 3 (below 320 feet at the top of the slope).

4.2.2 Soil Consolidation Design Parameters

Undrained shear strength profiles were calculated based on data from CPT-1-21 and advanced laboratory results, following the procedures outlined in Federal Highway Administration (FHWA) IF-02-034 and EPRI EL-6800. These procedures relate the stress path adjusted undrained shear strength to the stress history and effective stress parameters.

Exhibit 4-5, below, provides consolidation properties from the CRS testing completed on samples from H-1vw-20 and H-4si-21. Detailed plots of CRS test results are provided in Appendix C. The consolidation properties determined from H-1vw-20 and H-4si-21 seem to be consistent with those from CPT-1-21. Therefore, the linear consolidation properties from the CRS and CPTs are both what we are considering the “best estimate”. All results presented herein are using the consolidation properties from the CRSs, unless otherwise specified.

Exhibit 4-5: Project-Specific Soil Consolidation Properties

ESU #	Relevant Test	Test Depth (feet)	Preconsolidation Pressure (ksf)	OCR	e_0	C_c	C_r
2	H-4si-21, PS-6	10	13.7	10	0.42	0.18	0.015
2	H-4si-21, PS-12	20	15.1	5.81	0.41	0.16	0.012
2	H-4si-21, PS-18	30	14.4	3.69	0.45	0.2	0.015
3	H-1vwp-20, PS-23	76.5	16.4	1.93	0.47	0.23	0.017

NOTES:
1. ksf = kips per square foot

4.3 Surface Water and Groundwater

4.3.1 Surface Water

Squalicum Creek is the primary source of natural surface water in the vicinity of the project site. The creek generally flows from east to west, from Squalicum Lake to Bellingham Bay.

4.3.2 Groundwater

We anticipate the groundwater levels at the site location are primarily controlled by the seasonal variation in rainfall and, to a lesser extent, the Squalicum Creek elevation. One open standpipe piezometer was installed in boring H-2p-20 to monitor the groundwater levels. A pressure transducer with a Level Troll 500 data logger was installed in the piezometer to continuously monitor the groundwater level. Two VWPs were installed in borings H-1vwp-20 and H-3vw-20, to continuously monitor the groundwater levels within the upper and lower groundwater zones observed during drilling. The upper groundwater zone appears to be perched within the embankment fill atop the relatively impervious ESU 3.

The measurements collected from the piezometers cover the period between July 2020 to December 2021. The groundwater levels during this period generally fluctuated as shown in Exhibit 4-6. Plots of groundwater level readings from each transducer (along with rainfall data) are presented in Appendix D.

EXHIBIT 4-6: GROUNDWATER DATA SUMMARY

Boring	Ground Surface Elevation (feet)	Shallow Perched Water Elevation (feet)		Deeper Groundwater Elevation (feet)	
		Minimum	Maximum	Minimum	Maximum
H-1vw-20	323.0	293.0	297.5	200.0	204.5
H-2p-20	323.0	NA[1]	NA	270.5	274.0
H-3vw-20	323.5	301.5	312.0	204.0	205.5

NOTE:
1. NA = Not Applicable

We recommend a design groundwater elevation of elevation 272 feet, generally aligning with the normal high-water surface of the Squalicum Creek. This conservatively captures the range of deeper groundwater observed in the piezometers.

4.4 POTENTIAL VARIANCE

The subsurface interpretation and engineering analyses are based on the field exploration and laboratory testing program described previously. These interpretations are specific to the locations and depths noted on the boring logs (Appendix A) and subsurface profiles (Figures 4 and 5) and may not be applicable to all areas of the site. No number of explorations can precisely predict the characteristics, quality, or distribution of subsurface conditions. Potential variation from what is shown on the boring logs includes, but is not limited to:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (both natural and man-made) may result in changes to site and subsurface conditions.
- Groundwater levels at the site fluctuate seasonally due to precipitation or creek level and may be higher than measured during our monitoring period.
- SPT N-values in gravelly and cobble-rich soils may be unrealistic. Actual soil density may be lower than estimated from the N-values if the test was performed on a piece of gravel or cobble.
- Although not specifically encountered in the borings, boulders may be present in the Bellingham Drift unit (ESU 4).

If conditions different from those described herein are encountered during construction, we should review our interpretation and reconsider our geotechnical recommendations presented herein.

5 GEOLOGIC HAZARDS

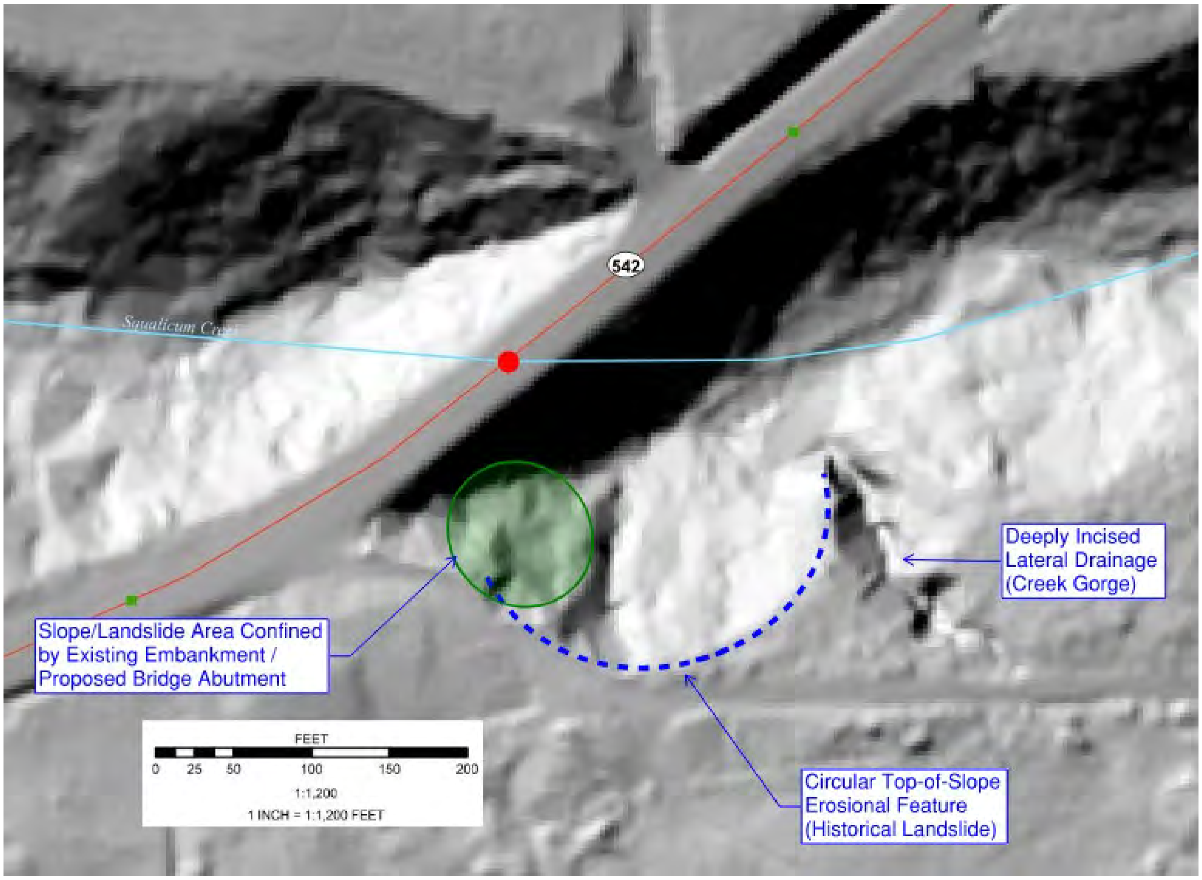
5.1 SCOUR

According to the information provided by the WSDOT Hydraulic Office, the maximum scour elevation for both the 100-year and 500-year flood events is at approximately 261.0 feet, based on a reconfigured creek thalweg elevation of 267.3 feet at the downstream face and 268.8 feet at the upstream face. During the design life of the bridge, the creek channel is assumed to migrate; therefore, this scour elevation is also assumed to occur at or near the proposed bridge abutments. The proposed design includes installation of a sheetpile wall at the base of the bridge abutments to protect the structures against potential scour.

5.2 LANDSLIDES

According to the DNR Geologic Hazard Maps (DNR 2019), no landslides have occurred at or near the site. However, LiDAR imagery (2017) indicates that historic landsliding may have occurred along the south creek bank just east of the proposed bridge structure. As can be seen in Exhibit 5-1, below, there are several large circular top-of-slope erosional features that are similar to what you would expect for a landslide head scarp.

EXHIBIT 5-1: SITE LIDAR IMAGE SHOWING SUSPECTED HISTORICAL LANDSLIDING



The imagery generally shows moderately sloped hillsides and gullies with lateral drainage features toward Squalicum Creek. These local changes in topography are likely associated with surficial scour and erosion from rainfall and runoff. However, the relatively large circular feature just southeast of the SR 542 roadway is considered a potential landslide impact risk to the proposed bridge. A similar feature further east is considered to be too far away to pose any meaningful risk to the structure. Between these two features, the LiDAR imagery shows the deeply incised lateral drainage channel that is labeled *Lateral Creek Gorge* elsewhere in this report.

Given the suspected past landsliding activity close to the proposed structure, an initial field reconnaissance was performed by GO staff on December 3, 2019, and two slope inclinometers were installed to measure potential slope movements (borings H-4si-21 and H-5si-21). These were installed in February 2021 and the data recorded to date are included in Appendix E. Haley and Aldrich Augmentation Staff (Licensed Geologist and Professional Geotechnical Engineer) subsequently made a site visit to perform a second field reconnaissance on November 5, 2021.

5.2.1 *FIELD RECONNAISSANCE*

During our field reconnaissance in November 2021, we made the following pertinent surface observations in the area of the suspected landslide:

- **Historical Landslide Headscarp.** We observed a 5- to 10-foot high, very steep to near-vertical, exposed soil face along the upper portion of the suspected landslide area, aligning with the large circular top-of-slope erosional feature observed on the LiDAR map. This is consistent with a headscarp feature and would support the presence of a historical landslide. This feature is much less pronounced or non-existent within the west half of the landslide area (adjacent to SR 542).
- **Hummocky Terrain and Tree Growth.** A hummocky terrain was observed within the slope area below the observed headscarp. Several large-diameter, straight trees (relatively old and living) were observed within the level or gently sloping area above the headscarp. Several trees within the slope area below were observed to be leaning or curved (pistol-butted). This indicates that the surficial soils within the landslide have been slowly creeping or moving downslope in the past.
- **Lateral Creek Gorge Soil Exposures.** The exposed soils within the deeply incised creek gorge visible on the LiDAR map (see previous section) were observed to consist of sandy Silt/Clay with Gravel and Cobbles. We observed the gorge sidewalls to be steep to near-vertical, indicating relatively competent material (stiff to very stiff). This is consistent with soil conditions observed in the landslide slope borings further west (H-4si and H-5si). Two surface samples of these exposed creek gorge soils were collected and analyzed for overall site soil correlation purposes. Their locations are depicted on Figure 2 and the lab test results are included in Appendix B.

5.2.2 *Slope Inclinometer Data and Undisturbed Soil Sample Review*

To assess how deep the historical landslide failure plane might be, we installed and monitored slope inclinometers in the slope borings (H-4si-21 and H-5si-21). After over a year of monitoring data, there is no indication of on-going deeper movement within the exploration depth (30 to 40 feet bgs). There is some indication of possible near-surface soil movement in the upper 5 to 6 feet bgs; however, the recorded displacement is very small (on the order of 0.1 inches) and may be due to something other than actual soil movement.

To supplement the inclinometer monitoring data, we also reviewed all undisturbed samples within the two slope borings for evidence of a disturbed soil zone that may be related to a landslide failure plane. The samples were extracted from Shelby tube cases and split by hand longitudinally so that fresh soil surfaces could be assessed for slip surfaces and abnormalities. Review indicated that samples generally consisted of homogeneous gray to brown Clay and Silt with Sand and occasional Gravel. No cavities or evidence of soil disturbance indicative of a landslide failure plane was observed in any sample. Coupled with the lack of any disturbed soil observation during SPT sampling in the field, we conclude that the historical landslide failure plane, if it still exists, is located deeper than 30 to 40 feet below the slope surface.

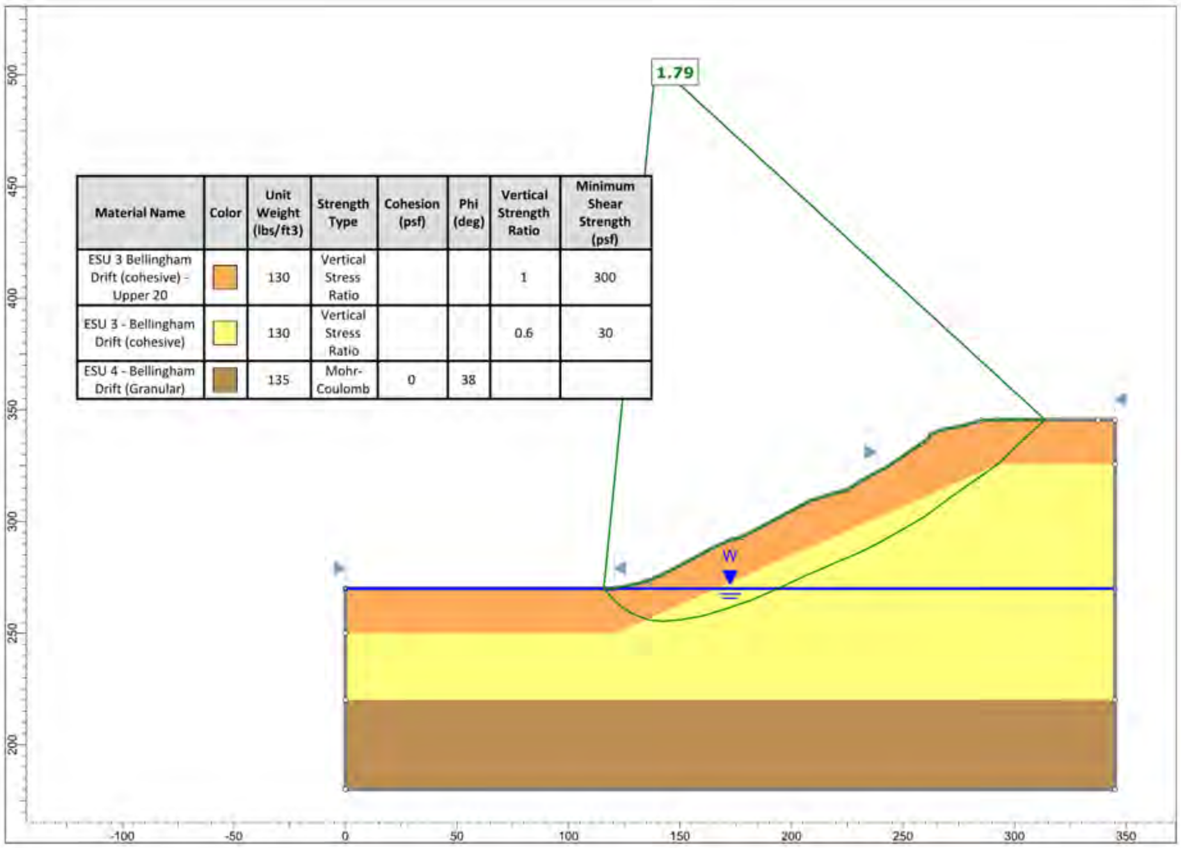
5.2.3 *Slope Stability Analyses*

To evaluate the potential for future slope movement, we performed two-dimensional, limit-equilibrium slope stability analyses of subsurface profile B-B' taken through the historical landslide area (Figure 5). The overall slope inclination is approximately 1.9H:1V (horizontal:vertical) and the slope height is about 60 feet. For simplicity, the slope was conservatively analyzed without the resistance provided by the existing or future embankment.

The slope stability analyses were performed using the SLIDE2 (v9.014) software package (Rocscience 2021). Factor of Safety (FS) against sliding failure was evaluated using the Spencer and Morgenstern-Price methods. Given the apparent lack of a distinct landslide failure plane within the upper 30 to 40 feet below the slope surface, we modeled the slope soils using the shear strength ratios recommended in *Section 4.2.1*. This is supported by the field exploration and advanced soil testing data, as well as the demonstrated consistency of soil properties within ESU 3 across the site.

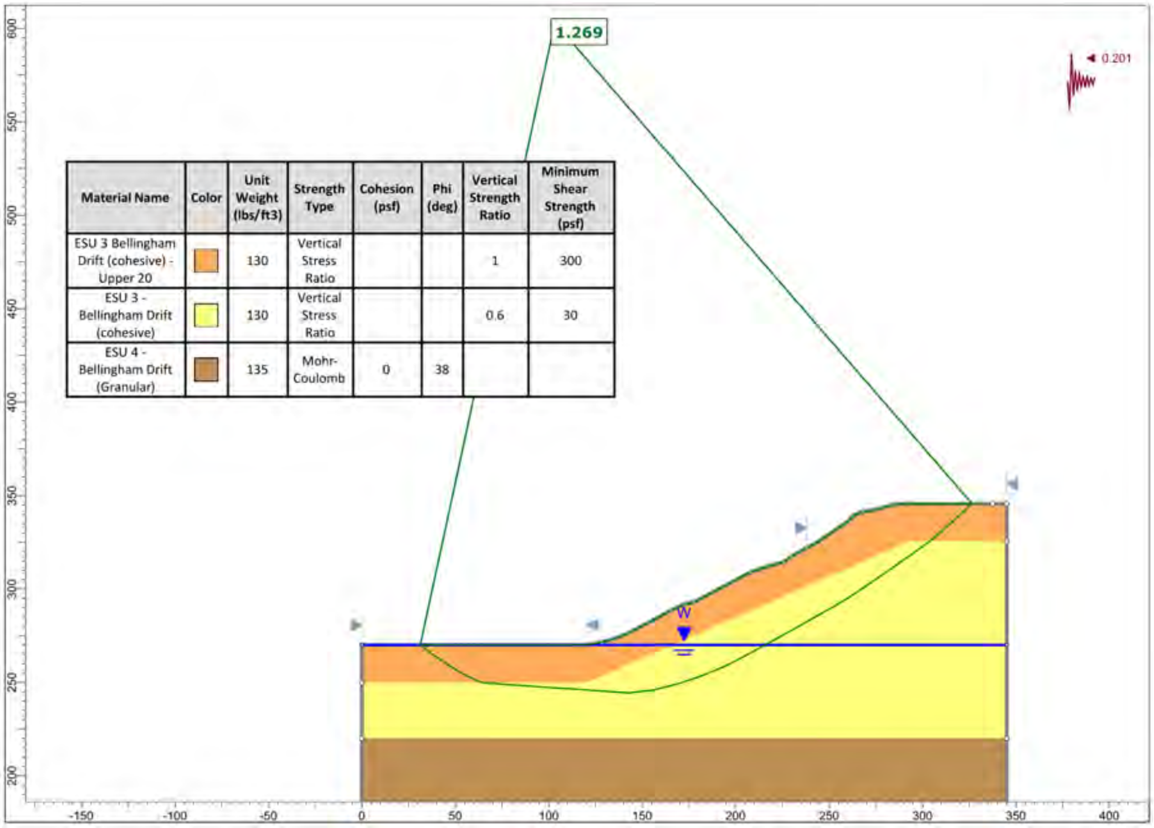
Our landslide slope stability analyses indicate the minimum static FS is 1.79. In the seismic case, the minimum FS is 1.27. The stratigraphy and soil parameters used, along with the most critical failure surface, is depicted below in Exhibits 5-2 and 5-3 for both the static and seismic design cases.

EXHIBIT 5-2: LANDSLIDE STATIC SLOPE STABILITY ANALYSIS RESULTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)
ESU 3 Bellingham Drift (cohesive) - Upper 20		130	Vertical Stress Ratio			1	300
ESU 3 - Bellingham Drift (cohesive)		130	Vertical Stress Ratio			0.6	30
ESU 4 - Bellingham Drift (Granular)		135	Mohr-Coulomb	0	38		

EXHIBIT 5-3: LANDSLIDE PSEUDOSTATIC SLOPE STABILITY ANALYSIS RESULTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)
ESU 3 Bellingham Drift (cohesive) - Upper 20		130	Vertical Stress Ratio			1	300
ESU 3 - Bellingham Drift (cohesive)		130	Vertical Stress Ratio			0.6	30
ESU 4 - Bellingham Drift (Granular)		135	Mohr-Coulomb	0	38		

Given that the field explorations/inclinometers do not indicate the presence of a near-surface historical landslide failure plane, we performed a FS=1.0 back calculation to estimate a most likely critical failure circle depth (assuming artificially weak soil strengths). This indicates that the historical landslide was likely a relatively deep rotational failure, with an exit point within the valley floor as much as 50 feet or more away from the bottom of the steep slope.

5.2.4 RISK ASSESSMENT

Based on the field exploration, inclinometer data, and slope stability analysis, it is our opinion that the risk of further landslide slope movement or failure potentially impacting the proposed structure is low. The following are the main reasons for this:

- The historical landslide is interpreted to have occurred before the construction of the existing SR 542 road embankment, which has essentially buttressed the western portion of the original slide area (See Exhibit 5-1). This has increased the resistance against sliding, effectively reducing the likelihood that the old slide failure plane can be reactivated. The proposed new southwest bridge abutment will continue to provide this confinement against lateral slope movement and potential reactivation of the western portion of the historical landslide area.

- While the inclinometer data show possible shallow near-surface soil creep, there is no indication of a landslide-induced weak failure plane within 30 to 40 feet below the slope surface.
- Potential reactivation of a deeper historical landslide failure plane would likely result in a rotational failure, which is not likely to generate a lateral debris flow that could impact the structure.
- If the old landslide failure is retriggered (for example by a seismic event), the majority of the failure mass would likely flow toward the remaining, opposite creek side slope just east of the proposed bridge structure.

Although the risk for future reactivation of the historical landslide is low, there is some risk that the landslide area slopes could be undermined if significant cuts are made along the slope bottom during construction. To reduce this risk, we recommend that any temporary cut slope to place new creek bed material along the base of the existing channel side slope not be steeper than 2H:1V. This will roughly match the overall inclination of the existing side slope and therefore roughly maintain the FS of our analysis section.

5.3 Seismic Hazards

We evaluated potential seismic shaking at the site in accordance with 2021 WSDOT Geotechnical Design Manual (GDM), which considers the design earthquake to be seismic shaking having a 7 percent probability of exceedance in 75 years (approximately 1,000-year return period). Our evaluation used data obtained from the U.S. Geological Survey (USGS) Unified Hazard Tool Dynamic Conterminous (USGS 2021).

Based on this, the expected peak bedrock acceleration having a 7 percent probability of exceedance in 75 years is 0.294g. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The effective peak ground acceleration (A_s) is determined by applying a site class factor to the peak bedrock acceleration, as outlined in *Section 6.2* of this report. Based on the deaggregation of the site seismic hazard, the mean magnitude is 6.88 and distance to rupture is 59.5 kilometers (km). The modal magnitude is 7.1 with a distance to rupture of 68 km.

5.3.1 Fault Rupture

The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge abutment or retaining wall. The American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) (2020) (AASHTO Design Specifications) requires near fault effects to be taken into consideration for bridges with active faults within 6 miles and for structures with a period greater than 0.5 seconds.

The USGS maintains information on faults and associated folds in the United States that are believed to be sources of earthquakes greater than magnitude 6 during the Quaternary (the past 1,600,000 years). We have reviewed this information and determined that the site is not located within 6 miles of any of the mapped fault zones, as depicted on Figure 6. Given this, it is our opinion that the risk of fault rupture surface effects at the site is low.

5.3.2 Liquefaction and Cyclic Strength Loss Potential

Soil liquefaction is a phenomenon whereby saturated soil deposits temporarily lose strength and behave as a viscous fluid in response to cyclic loading. Soil types considered at the highest risk of liquefaction during a seismic event are loose to medium dense sandy soils. Since the site subsurface conditions generally consist of fine-grained soil underlain by very dense coarse-grained material, we performed an analysis to determine the liquefaction susceptibility using plasticity index and moisture content versus liquid limit, according to the method developed by Bray and Sancio (2006). Based on our analyses, the site soils are considered non-liquefiable or moderately susceptible to liquefaction.

We also performed a liquefaction assessment using the CPT data from exploration CPT-1-21. We utilized the software CPeT-IT and CLiq which employs Idriss and Boulanger 2014 (Idriss and Boulanger 2014) triggering criteria. We assumed an Ic cutoff value for clay-like material of 2.6. The results of the CPT analysis generally suggest no widespread liquefaction, with only a few thin soil layers showing liquefaction susceptibility (i.e., a FS less than 1).

Post-liquefaction (or reconsolidation) settlement occurs because fine-grained soil tends to consolidate after cyclic shaking. The ground surface settlement is not typically uniform across an area and can result in significant differential settlement. Based on the fine-grained nature of the native soils and the negligible risk of liquefaction, we do not anticipate post-liquefaction settlement to be a significant hazard to the project. However, we do consider the native fine-grained soils to be susceptible to cyclic softening and strength loss. We performed two paired DSS and CDSS tests on two undisturbed samples to evaluate the strength loss for the post-cyclic scenario. Tests results are shown in Appendix D. The test results suggest between 0 and 20 percent strength loss following 20 cycles of shaking at the design cyclic stress ratio. Based on this we used a 20 percent strength loss when assessing the post-cyclic stability of the bridge structure.

5.3.3 Lateral Spreading

Our analyses indicate the potential for lateral spreading affecting the proposed structures at the site is low, as long as our recommendations are followed. Although the soils below the existing embankments and immediately surrounding area may be susceptible to post-cyclic strength loss, the planned sheet pile foundations, MSE walls and Bridge Footings are not expected to be adversely impacted by this, based on the results of our slope stability analyses presented in the following sections.

6 GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

The following sections describe the engineering analyses and geotechnical engineering recommendations for the proposed bridge shallow foundations and abutments, sheet pile walls, and associated MSE walls. Our recommendations are based on the project requirements and the preliminary plans from the BSO dated April 2022, our discussions with the PEO and BSO, and our interpretation of the subsurface conditions shown on Figures 4 and 5 and described herein.

We have prepared our design recommendations considering the Project configuration as described herein. If the PEO or BSO develops additional or revised information about final foundation and wall configuration or other factors, the recommendations presented herein may need to be revised. The GO must be made aware of the revised or additional information so we can evaluate our recommendations for applicability.

For purposes of our analyses, it was necessary to assume the soil and groundwater conditions observed in the current explorations described in this report are representative throughout the Project site. However, subsurface conditions should be expected to vary (see *Section 4.4*). We may need to revise our recommendations during construction if different conditions are encountered in unexplored areas of the site.

6.1 Design Criteria and General Considerations

This project will be designed in accordance with the GDM (WSDOT 2021b), the WSDOT Bridge Design Manual (BDM) (WSDOT 2020), and the AASHTO LRFD Bridge Design Specifications (BDS) (AASHTO 2020). Based on these documents, the following specific design criteria and conditions/assumptions were considered when performing our analyses:

- Structures requiring seismic design shall meet the Safety Evaluation Earthquake performance level objectives of no-collapse, as described in GDM Section 6-1.2.1.
- The proposed bridges will be supported on shallow foundations bearing on a MSE abutment composed of low-density cellular concrete and welded wire mesh reinforcement.
- A sheetpile wall facing will be installed around the perimeter of the lowest MSE abutment tier, primarily to provide scour protection and a water seal to prevent LDCC saturation.
- Foundation elements shall be designed for the effects of potential 100- and 500-year event scour levels (BDM Section 8.3.3.D).
- Although temporary sloping is the responsibility of the Contractor and will depend on field conditions encountered during construction, we have assumed a conservative 1.5H:1V temporary slope inclination (steepest standard slope allowed by code), where applicable in our analyses. If needed for PEO planning purposes and to estimate quantities prior to construction, we recommend assuming this temporary sloping condition will be feasible or consider a need for structural shoring.

Additional analysis-specific criteria are referenced in subsequent sections.

6.2 Site Seismic Design Parameters

The ground shaking hazard can be defined in general terms using an appropriate acceleration response spectra and site coefficients, or by using a site-specific procedure.

In the general procedure, the spectral response parameters are determined using the 2014 Seismic Hazard Maps produced by the USGS depicting probabilistic ground motion and spectral response for 7 percent probability of exceedance in 75 years.

Based on AASHTO LRFD BDS (2020) Article 3.10.3.1, a site-specific response spectrum procedure is required when Site Class F conditions exist at the site (deep, soft clay soils and peat).

Using the above procedures, we classify the site soils below the existing embankment as Site Class E. In accordance with GDM Section 6-3, we recommend the seismic coefficients provided in Exhibit 6-1.

Exhibit 6-1: Seismic Design Parameters

Parameter	Recommended Value
Site Class Based on Soil Conditions	Site Class = E
Mean Magnitude	M = 6.9
Modal Magnitude	M = 7.1
Peak Ground Acceleration (PGA) Coefficient of Class B/C Rock Boundary	PGA = 0.294g
0.2-Second Period Spectral Acceleration Coefficient on Class B/C Rock Boundary	S_s = 0.664g
1.0-Second Period Spectral Acceleration Coefficient on Class B/C Rock Boundary	S_1 = 0.194g
Site Coefficient for the Peak Ground Acceleration Coefficient	F_{pga} = 1.619
Site Coefficient for 0.2-Second Period Spectral Acceleration	F_a = 1.438
Site coefficient for 1.0-Second Period Spectral Acceleration	F_v = 3.357
Effective Peak Ground Acceleration Coefficient (g)	$A_s = F_{pga}*(PGA)$ = 0.475g
Design Earthquake Response Spectral Acceleration Coefficient at 0.2-Second Period	$S_{DS} = F_a*S_s$ = 0.954g
Design Earthquake Response Spectral Acceleration Coefficient at 1.0-Second Period	$S_{D1} = F_v*S_1$ = 0.65g

6.3 Bridge Abutments

Based on the 90 percent bridge plans, the proposed bridge will consist of a single, approximately 105-foot-long structure supported on concrete abutments. The bridge is planned to be 46 feet wide and span Squalicum Creek. The layout of the proposed bridge structure is depicted on Figure 7.

A large amount of earthwork will be performed for this project and significant embankment fills will be removed along the existing roadway alignment. The new roadway alignment generally runs through the existing roadway. As discussed in *Section 4*, the native subsurface soil conditions at the site consist of a geologic formation known as the Bellingham Drift, which has been broken into ESUs 3 and 4. ESU 3 is prone to consolidation settlement and post-cyclic strength loss. To minimize this risk and other issues, we recommend the use of a lightweight backfill within the MSE abutment structure.

Although a number of lightweight fill options are available, LDCC is recommended to minimize the risk of excessive upward buoyancy forces, which would be significantly higher for synthetic foam-based products (such as Geofoam). The use of an MSE-LDCC abutment structure will eliminate the need to construct deep foundations or ground improvement to support the structure and associated retaining walls. This approach will also mitigate settlement issues and deep-seated slope stability/bearing failures.

6.3.1 Low-Density Cellular Concrete (LDCC)

The California Department of Transportation (CalTrans) classifies LDCC into six different classes based on the cast unit weight. The classes have cast unit weights ranging from about 24 pcf (Class 1) to 120 pcf (Class 6). WSDOT has typically adopted the use of LDCC Class 2 through 4. The most used densities of LDCC range from 27 pcf to 30 pcf, corresponding to Class 2.

For this project, Class 2 LDCC will generally be used as backfill within most of the reinforced zone. However, the stronger Class 4 LDCC is required within 2 feet of pavement subgrade in areas where heavy traffic loading is expected and within 4 feet below the bridge footings. Class 3 LDCC is recommended to provide additional support from 4 feet to 12 feet below the bridge footings, underlying the Class 4 LDCC. These LDCC limits are graphically shown on Figure 8.

The cast densities and minimum compressive strengths for the recommended Classes of LDCC are provided in Exhibit 6-2. These class properties mirror the physical properties for engineered fill provided in Table 3.2 of the American Concrete Institute's publication 523.1R-06 (ACI 2006): Guide for Cast-in-Place Low Density Cellular Concrete.

Exhibit 6-2: LDCC Properties

LDCC Class	Maximum Cast Density (pcf)	Minimum 28 Days of Compressive Strength (psi)
2	30	40
3	36	80
4	42	120

NOTE:

1. psi = pounds per square inch

LDCC is typically placed in limited thickness lifts of 2 feet or less, to reduce the potential for material compression (collapse of entrained air) during curing. Welded wire fabric (WWF) reinforcement is also placed as part of each lift to provide internal strength within the overall LDCC prism, creating the modified MSE support structure proposed for this project.

The proposed design is to place the LDCC in 2-foot-thick lifts. The cellular concrete supplier should provide recommendations regarding specific waiting periods between lifts, but for estimating purposes, a waiting period of approximately 12 hours per lift can be assumed.

To mitigate potential uplift from buoyant forces, the minimum density of all types of LDCC should not be less than 21 pcf.

It is important to note that cellular concrete should not be placed in standing water.

6.3.2 MSE-LDCC Abutment Design Considerations

The temporary cut slope, as shown on Figure 8, should be constructed with an overall slope ratio no steeper than 1.5H:1V. We have shown a stepped excavation pattern to key the abutment into the existing soils, corresponding to a typical WWF length increase of between 6 and 9 feet for each tier.

Figures 8 through 10 show the recommended WWF configurations and embedment lengths at the abutments. The WWF should be placed at vertical spacings of no greater than 2 feet and should be bent to act as a form to terminate the LDCC lifts at the abutment face. We recommend that the abutment faces, in profile and section, be battered at a minimum of 48V:1H (vertical:horizontal). This can be achieved by incorporating the batter into the bend or by using lift setbacks. The abutment faces should be covered with a permanent fascia (e.g., shotcrete) to protect the WWF wire and the LDCC. Additionally, we recommend construction of a 5-foot-thick concrete seal at the bottom of the abutment profile as shown in Figure 8. The purpose of the seal will be to counteract uplift pressures, limit the infiltration of groundwater into the LDCC system from the underlying subgrade, and provide subgrade protection during construction.

The MSE wall stability analysis was performed using the MSEW+ software package (ADAMA Engineering, Inc. 2021). Capacity Demand Ratio (CDR) against connection strength, pullout, direct sliding, and eccentricity failures were evaluated using the AASHTO 2017-2020 Simplified Method with complex geometry. Our analysis has identified two types of reinforcement necessary for the structures, W4.5XW3.5-8X12 and W7XW4-8X12. The properties of the WWF are shown in Exhibit 6-3, below. We assumed a drained friction angle of 40 degrees with zero cohesion as strength properties in our model for the LDCC backfill for the structure.

EXHIBIT 6-3: WWF PROPERTIES

Welded Wire Fabric	Minimum Longitudinal Wire Diameter (inches)	Minimum Transverse Wire Diameter (inches)	Minimum Longitudinal Wire C-C Spacing (inches)	Minimum Transverse Wire C-C Spacing (inches)	Minimum Galvanized Thickness (millimeters)	Length (feet)
W4.5XW 3.5-8X12	W4.5 (0.239)	W3.5 (0.213)	8	12	3.4	Varies
W7*W4-8X12	W7 (0.299)	W4 (0.215)	8	12	3.4	Varies

The WWF should conform to WSDOT General Special Provision 6-13.2.INST1.GR6 in that the fabrication of the WWF should occur prior to the galvanizing process. The WWF should conform to the specifications provided in ASTM A 82.

Transverse reinforcement should extend the full length of the exposed portions of the wall until it is embedded a minimum of 2 feet below final grade.

Since the three-sided abutments require reinforcement to be placed in three directions, the reinforcement will be somewhat congested. We recommend that the WWF be placed at the same elevations all the way around to act as formwork for the pours. We further recommend that spacers be used between the perpendicular reinforcement with respect the abutment wall faces to maintain a minimum separation of 2-inches between the reinforcement layers. This minimum separation will not be possible at the wall face where the perpendicular reinforcement may come into contact. This minimum contact between perpendicular layers is acceptable. The WWF mats may require mechanical connections by means of welds to transverse wires to minimize relative movement between adjacent mats. This has been detailed in Figure 11.

With respect to the bent WWF acting as a form for the LDCC, our recommendation is to use a construction geotextile to prevent the LDCC from flowing through the face, as detailed in Figure 11. For this, we recommend using a geotextile for ditch lining (woven) conforming to Table 4, Section 9-33 of the Standard Specifications. The BSO will need to determine if the bent mats and geotextile can support the fluid pressure of the wet LDCC. If not, a bracing system will need to be designed and shown in the plans.

The MSE-LDCC abutments have been designed as walls, and its outside faces should therefore be considered as such. To maintain adequate stability, each wall face should have a minimum of a 4-foot-level bench in front, with a toe slope ratio no steeper than a 2H:1V. The walls further need to be embedded into the sheetpile walls with a minimum of 2 feet or 10 percent of the total wall height, whichever is greater. This has been detailed in Figure 8.

6.3.3 MSE-LDCC GLOBAL STABILITY

We performed global stability analyses of the planned MSE-LDCC Abutments using limit equilibrium methods and the SLIDE computer software. SLIDE performs two-dimensional limit equilibrium analysis to analyze slope stability by determining a FS against global failure. We performed analyses for the strength limit, and extreme event limit states, including analyses for pseudo static (seismic), post seismic reduced strength conditions, a 500-year scour event, and a coupling event (scour and pseudostatic). For the coupling scenario, full seismic loading was used, and the scour elevation was increased to 264.5 feet to reflect 50 percent of the total scour design flood depth. Our analyses indicate that the planned MSE-LDCC Abutments meet the FS requirements for global stability specified in the GDM (WSDOT 2021b).

The MSE-LDCC Abutments were modeled with an approximate 46-foot exposed wall height, extending from the bottom of pavement elevation 320 feet down to the top of sheet pile elevation 274 feet, and an approximate 13-foot embedment below the top of the sheet piles. The sheet piles consist of a 13-foot exposed face from elevation 274 feet down to scour elevation 261 feet, and an embedment of 27 feet to the design tip elevation of 234 feet. The sheet piles are designed to protect the MSE-LDCC structure from scour and act as groundwater cutoff both in the temporary excavation and permanent design case. The LDCC was modeled with the WWF reinforcement described in *Section 6.3.2*. The WWF reinforcement members were embedded from the wall face to lengths of approximately 0.4, 0.55, 0.7, and 0.8 times the design height of the abutment wall.

Drawings depict final grade above the MSE-LDCC structure will be at approximately elevation 323.0 feet. The ground surface at the base in front of the structure was set at approximately elevation 260.5 feet, to depict scour in the event of a 100-year flood (i.e., the design scour event). Results from SLIDE indicate the minimum static FS is 1.38. Models were also developed for pseudostatic, residual strengths, scour from a 500-year flood, and a coupling event with pseudostatic loading and scour. Analysis of these models indicate that all the extreme event limit state cases exceed the minimum FS requirements (1.1), using the Spencer and GLE/Morgenstern-Price methods.

An additional model was developed to evaluate temporary stability during construction. This temporary construction scenario was modeled with one sheet pile providing reinforcement to the hillslope prior to construction of the MSE-LDCC structure. We used the same design parameters for the sheet piles as before, with the top of pile elevation at 274 feet, base of excavation within the sheetpiles at elevation 261 feet, and 27 feet of embedment to the pile tip. The sloped soil from the top of the sheetpile to the bottom of pavement elevation was set at a 2H:1V to simulate the required temporary excavation slope. Analysis results for this model indicate that the temporary construction FS exceeds the minimum design requirement (1.33).

6.3.3.1 Transverse MSE-LDCC Global Stability

We performed global stability analyses transverse of the planned MSE-LDCC structures using limit equilibrium methods and the SLIDE computer software. We understand that site constraints require the north side slope at approximately Station 10+40 to be approximately 1.5H:1V and transition to 2H:1V by Station 10+80. Permanent slopes for other portions of the project are at 2H:1V. We selected three sections (stationing STA 10+40, STA 10+70, and STA 13+10) for transverse stability analysis. These sections were selected based on the proposed grading and distances from the top of slope.

Slope stability models for transverse failure were constructed based on cross-sections provided by the PEO. These sections were used to determine adjacent grading and the MSE-LDCC embedment at different points in the alignment. The models incorporate a factored traffic load to replicate loading at the top of the MSE structure along with the design groundwater elevation that is expected after construction. Results for transverse slope analyses indicate the extreme event limit states achieve the minimum required FS for the project for the critical sections analyzed. Results for the strength limit states meet the minimum required FS for adjacent slopes graded at 2H:1V. For the proposed slopes steeper than 2H:1V the results indicate the potential for shallow failures. While these failures do not impact the structure, the project office should anticipate the need for maintenance to address any shallow sloughing failures throughout the design life of the structure.

6.4 Bridge Shallow Foundations

6.4.1 Bearing Resistance

The bridge spread footings will be supported directly on Class 4 LDCC material, with a 28-day compressive strength of 120 psi. We performed an analysis to determine the nominal bearing resistance that may be used to design these bridge foundations under the service, strength, and extreme event limit state conditions. The nominal bearing resistance values for the bridge footings are provided in Exhibit 6-4. The resistance factors in *Section 6.4.2* should be applied to these nominal bearing resistance values to determine factored resistances.

Exhibit 6-4: Bridge Foundation Nominal Bearing Resistance

Footing Type	Bearing Soils	Footing Size (L x B)	Nominal Bearing Resistance (ksf)
Bridge Shallow Foundations	Class 4 LDCC	37’-4” X 10’-6”	17

The recommendations presented in Exhibit 6-4 are greater than those prescribed in GDM Section 15-5.3.6 for bridge foundations placed directly above an MSE wall reinforced zone. The compressive strength of the LDCC backfill material provides a greater bearing capacity than the standard gravel borrow for structural earth walls backfill.

We are designing the bridge abutment/embankment to limit the amount of anticipated settlement by using the LDCC as backfill. However, we would still estimate up to 1 inch of total settlement and up to ½ inch of differential settlement between the bridge foundations, due to compression of the embankment material and settlement of the underlying soils. To reduce the risk of potential LDCC edge failure, we recommend assuming a setback distance from the abutment face to the edge of the bridge footing of between 4 and 6 feet, as shown in Figure 8.

6.4.2 Resistance Factors

Resistance factors were determined using the GDM Chapter 8 and AASHTO Design Specifications Article 10.5. Resistance factors for bearing resistance at each respective limit state are presented in Exhibit 6-5, below.

Exhibit 6-5: Spread Footing Resistance Factors for Foundations

Resistance Factor, ϕ		
Strength	**Service**	**Extreme Event**
0.45	1	0.9

6.4.3 Foundation Spring Stiffness (Shear Modulus Ratio – G/G_0)

Footing deflection, or spring stiffness, may be estimated using an elastic stress computation method and an estimated elastic modulus for the material beneath the foundations, per Section 6.2.2.1 of the Federal Highway Administration (FHWA) – Seismic Retrofitting Manual for Highway Structures (FHWA 2006). We estimated the shear modulus, G, using equation 6-1 of the FHWA Seismic Retrofitting Manual for soils within a distance of 2*B of the bottom of the bridge footings. Based on this analysis, we recommend a G of 35,000 ksf based on the elastic modulus of Class IV LDCC. Based on Table 6-7 of the GDM, we recommend a Poisson's ratio (v) value of 0.15 for Class 2 LDCC.

6.5 Sheetpile Recommendations

Sheet piles should be designed such that they can withstand earth pressure and loading conditions outlined during the temporary construction and permanent phase. Design earth pressures for the temporary construction phase, including construction surcharge, are provided in Figure 12. Earth pressures for the permanent condition, including traffic surcharges, seismic loads, and the LDCC abutment, are provided in Figure 13.

Furthermore, to reduce the risk of water infiltration and potential upward buoyant forces within the LDCC, we recommend waterproofing the sheetpile system. The waterproofing should be designed to maintain the seal through the design life of the structure.

We understand an additional sheetpile wall will be installed for the project extending from the west abutment in the upstream direction and positioned at the base of an approximately 2H:1V slope. The purpose of the sheetpile wall will be to protect the project from lateral migration as identified in the scour assessment. We do not recommend designing this scour protection sheetpile system for any surcharge or seismic loading conditions. Figure 15 attached presents earth pressure recommendations for the design of this wall.

6.6 Approach Slabs

Following discussion with the State Geotechnical Engineer, the BSO, and the PEO, we understand that approach slabs will not be required for this project. This is primarily due to the lightweight nature of the proposed abutment structure and the resulting negligible estimated settlements post-construction.

Per Section 8.6.5.3 of the GDM, approach slabs may be deleted for geotechnical reasons if any of the following conditions are met:

- If settlements are excessive, resulting in the angular distortion of the slab to be great enough to become a safety problem for motorists, with excessive defined as a differential settlement between the bridge and the approach fill of 8 inches or more;
- If creep settlement of the approach fill will be less than 0.5 inch, and the amount of new fill placed at the approach is less than 20 feet;
- If approach fill heights are less than 8 feet; or
- If more than 2 inches of differential settlement could occur between the centerline and shoulder.

At the design bridge location, the anticipated amount of creep settlement will be less than 0.5 inch, and the amount of new soil backfill placed at the site is less than 20 feet (not including LDCC). Therefore, bridge approach slabs can be deleted for geotechnical reasons. However, final removal of bridge approach slabs must also be approved by the State Geotechnical Engineer and State Bridge Design Engineer, per Section 720.03(8) of the WSDOT Design Manual (WSDOT 2020b). We understand that the PEO has obtained this final approval.

6.7 General Backfill Requirements

All backfill placed as part of the abutments shall be placed as LDCC, as described in *Section 6.3.1*. Any backfill outside of the prescribed MSE-LDCC Abutment and excavation area, as presented in the plans and specifications, should be compacted Select Borrow (WSS Section 9-03.14(2)) or Gravel Borrow (WSS Section 9-03.14(4)) using Method B, as described in WSS Section 2-03.3(14)B and 2-03.3(14)C.

6.8 Permanent Cut Slopes

Permanent cuts of the existing embankments are planned at the Squalicum Fish Passage bridge. We recommend designing cut slopes for a maximum slope angle of 2H:1V. Until a layer of vegetation is established, the upper 1 to 2 feet below the surface of the slope may be only marginally stable. We recommend that measures be taken to control erosion on new permanent slopes. Such measures should include both short-term and long-term strategies for erosion control. The design of these erosion control measures will be performed by others.

7 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

The project will be constructed per the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSS) (WSDOT 2022). We have developed construction considerations for the project to assist in preparation of Special Provisions and to identify key geotechnical issues that should be prepared for and observed during construction.

Our recommendations are not intended to dictate methods or sequences used by Contractors. Prospective Contractors must undertake their own independent review and evaluation of the subsurface data to arrive at decisions concerning the planning of the work; the selection of equipment, means and methods, techniques, and sequences of construction; establishment of safety precautions; and evaluation of the influence of construction on adjacent sites.

The following sections describe recommendations for subgrade preparation, considerations for temporary slopes and shoring, and re-use of excavated material recommendations for embankment and structural backfill. Our explorations performed for the project may not be sufficient for design of temporary slopes and shoring. It is the responsibility of the Contractor to conduct additional explorations if needed for the design of their temporary works.

7.1 Bridge Abutment and Foundation Subgrade Preparation

The bridge foundations will bear on the LDCC abutment, as previously described. The Contractor should confirm that the LDCC subgrade has properly cured prior to construction of the shallow foundations.

The subgrade within the MSE-LDCC Abutment structure and sheetpile concrete seal area should be carefully prepared and protected before LDCC/concrete is placed. All loose debris, organic, or otherwise unsuitable material shall be removed from beneath concrete seal or any LDCC/concrete fill areas. Any loosening of the materials during construction could result in larger settlements than those estimated. It is important to clean the subgrade excavation of loose or disturbed soil and that there be no standing water before placing any LDCC. Also, groundwater should be controlled so the subgrade does not boil, heave, or become loosened due to groundwater seepage. These conditions should be documented during construction.

A WSDOT geotechnical inspector should review and approve all LDCC/concrete and other embankment fill subgrades, including over-excavation and placement of aggregate base material and geotextile separation fabric, if needed.

7.2 Temporary Slopes and Shoring

Temporary slopes and/or shoring will be necessary to construct various elements included in this project. Temporary slopes and shoring are the responsibility of the Contractor, and the Contractor will determine the appropriate measures to ensure that all excavation work is in compliance with local, state, and federal safety codes, and in accordance with the requirements in the GDM. The Washington Administrative Code (WAC) 296-155 contains specific requirements for trenches and temporary slopes, as do the WSS and GDM. Any construction slopes discussed herein are for design/planning purposes only and should not be interpreted as a direction of what will constitute safe slopes in the field during construction.

Where groundwater seepage is encountered, erosion could occur such that the stability of temporary excavation slopes is adversely affected. The Contractor should be prepared to control groundwater seepage and prevent erosion that could cause slope instability.

Depending on the space or alternative routes available for traffic diversion around the excavation, temporary shoring walls may be required to allow for excavation and staging the work. Shoring walls may also be required to protect or excavate to install utilities. Where shoring walls are required to support SR 542 traffic, structural shoring is required. WSS Section 2-09.3(3)D (also refers to the GDM) provides design and construction requirements for structural shoring. The design of any temporary shoring proposed for this project should be in accordance with Washington Department of Occupational Safety and Health (DOSH) and GDM guidelines and should be reviewed by the GO and the BSO.

7.3 Reusing Existing Embankment Material

The existing embankment material excavated during construction will likely not be suitable for reuse as backfill due to the high fines content. Some of the excavated material may be reused if found to be suitable based on grain size laboratory testing, and it meets the material requirements of Gravel/Select Borrow. It must also meet moisture requirements for compaction. Moisture conditioning may be necessary depending on the moisture content of the material when excavated and stockpiled. If construction will occur during wet weather, we recommend using Gravel Borrow rather than the existing embankment material. The Contractor should stockpile and protect suitable excavated embankment material with plastic sheeting.

7.4 Dewatering for Structure Excavations

Based on the groundwater levels measured in test borings at the time of drilling and subsequent measurements form the piezometer data loggers (see *Section 4.2*), we do not anticipate the planned bridge foundations will extend below the current groundwater level. Perched groundwater was observed within the existing roadway embankments at the time of drilling. If perched groundwater is observed during construction, groundwater and surface water flowing into the excavation area should be routed away from the excavation area to an appropriate location where it can be treated (if necessary) and discharged.

We do anticipate the need for construction dewatering for the excavation within the sheetpile system. The use of sump pumps may prove feasible for dewatering excavations given the low permeability of the native subgrade soils. However, depending on construction staging/sequencing, the Contractor may need to install well points prior to the excavation. We, therefore, recommend including a Special Provision for dewatering in the contract.

Dewatering is the responsibility of the Contractor, who is solely responsible for construction means and methods and site safety. The Contractor should select, design, construct, and operate the dewatering system, in conjunction with the Contractor's design and construction of the excavation/shoring system. The dewatering design and construction plans should be submitted for review and approval by the GO and the Bridge and Structures Office.

7.5 Geotechnical Instrumentation and Well Abandonment

7.5.1 MSE-LDCC Instrumentation monitoring – Construction and Long Term

Recommendations for long term and construction monitoring of the MSE-LDCC Abutment reinforcement would be provided under a separate memorandum.

7.5.2 Well Abandonment

All test holes with open standpipe piezometers will need to be decommissioned in accordance with WAC 173-160. Typically, this will occur after one year of monitoring, but may, in some cases, be earlier if project construction schedule requires this (as long as seasonally high groundwater levels have been recorded). The GO will generally coordinate this effort, but with input from the PEO project manager regarding construction schedule and potential other than geotechnical reasons to keep wells in place longer. If applicable, please contact the Geotechnical design engineer for this project/report to discuss further. Without project-specific considerations, test hole abandonment should ideally occur prior to contract advertisement.

7.5.3 Slope Inclinometer Monitoring and Casing Abandonment

Although no significant inclinometer movement has been observed to date, it is possible that long-term soil movement or creep could still be occurring within the suspected landslide area. Therefore, we recommend continued monitoring of the inclinometers through the construction phase (on a monthly basis). If movement is occurring, this will allow us to note the depth and rate of movement, which are key factors to better understand the nature of the movement and potential risk to the project.

8 RECOMMENDED ADDITIONAL SERVICES

The future performance and integrity of the structural and geotechnical elements of the Project will depend largely on proper preparation of the PS&E documents and diligent construction procedures. Therefore, we recommend that the GO in conjunction with the Regional Materials Engineer (RME) provide the following post-report services:

- The GO Should prepare the Summary of Geotechnical Conditions to be included as an appendix to the Special Provisions. The summary should be prepared as part of the PS&E review process.
- The GO and RME should review all construction plans and specifications to verify that the design criteria presented in this report have been interpreted correctly and properly integrated into the design.

- The GO and RME should attend pre-construction conferences with the Construction Project Engineer and Contractor to discuss important construction-related issues.
- The GO and RME should Review Contractor submittals for all shoring walls or temporary slopes, permanent walls, and other geotechnically challenging elements of the Project.
- The RME should observe the following to confirm that geotechnical requirements are met:
    - Exposed subgrade for retaining wall foundations after completion of stripping and excavation to contract elevations.
    - Placement and compaction of LDCC, structural fill, and backfill.

In addition to the aforementioned services, the GO can provide inspector training, assist in change of conditions claims, and review value engineering design change proposals.

9 CLOSURE

This geotechnical report was prepared to summarize the recent explorations, laboratory tests, and engineering analyses, and to provide final design recommendations and construction considerations for the SR 542 Squalicum Creek Fish Passage project. This report should not be used for other purposes without contacting the GO for a review of the applicability of such reuse. This report should be made available to prospective Contractors for their information or factual data only and not as a warranty of ground conditions.

10 REFERENCES

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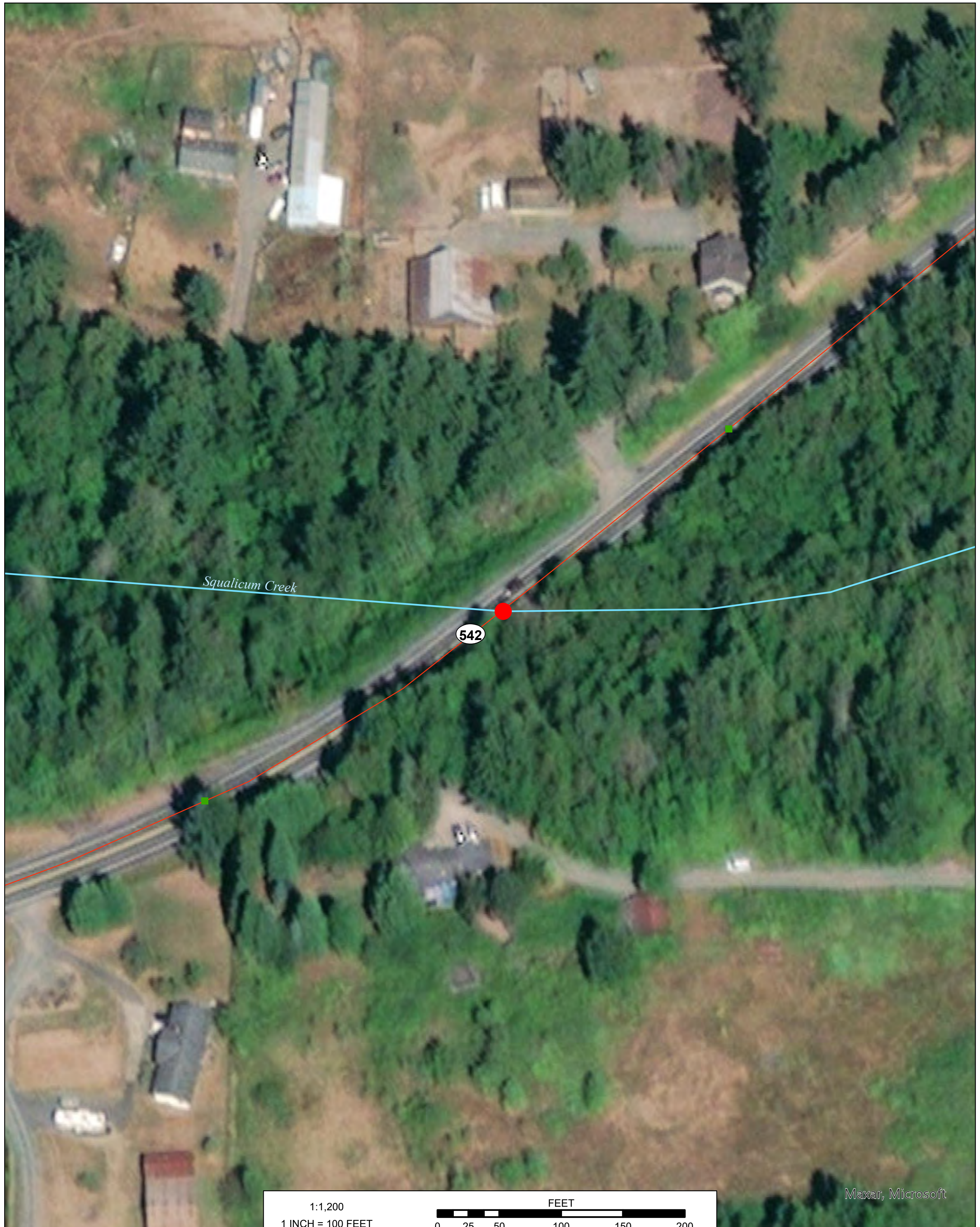
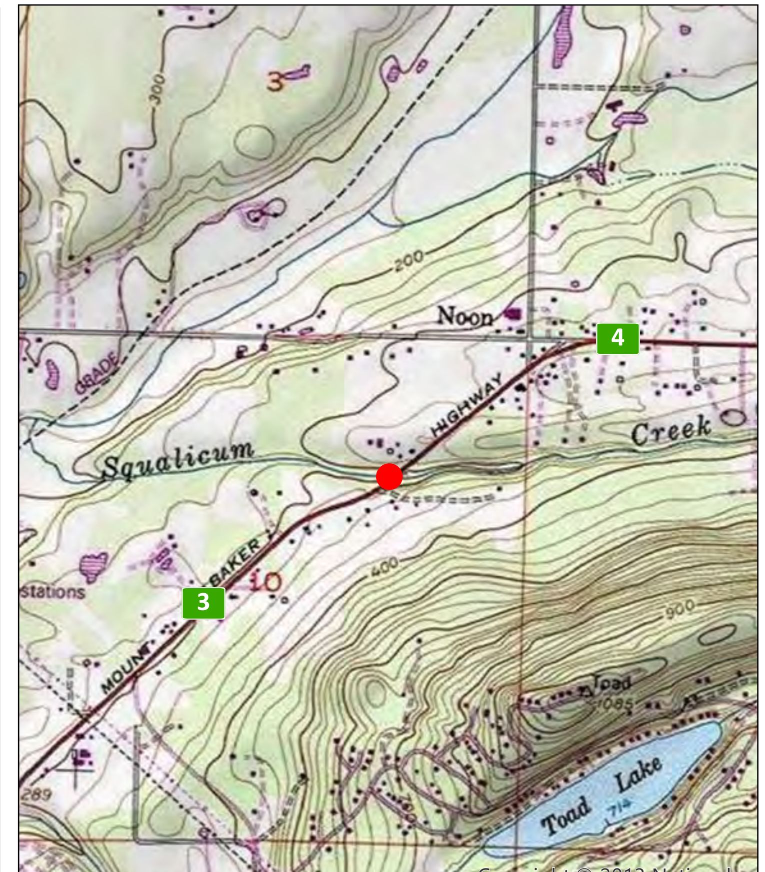


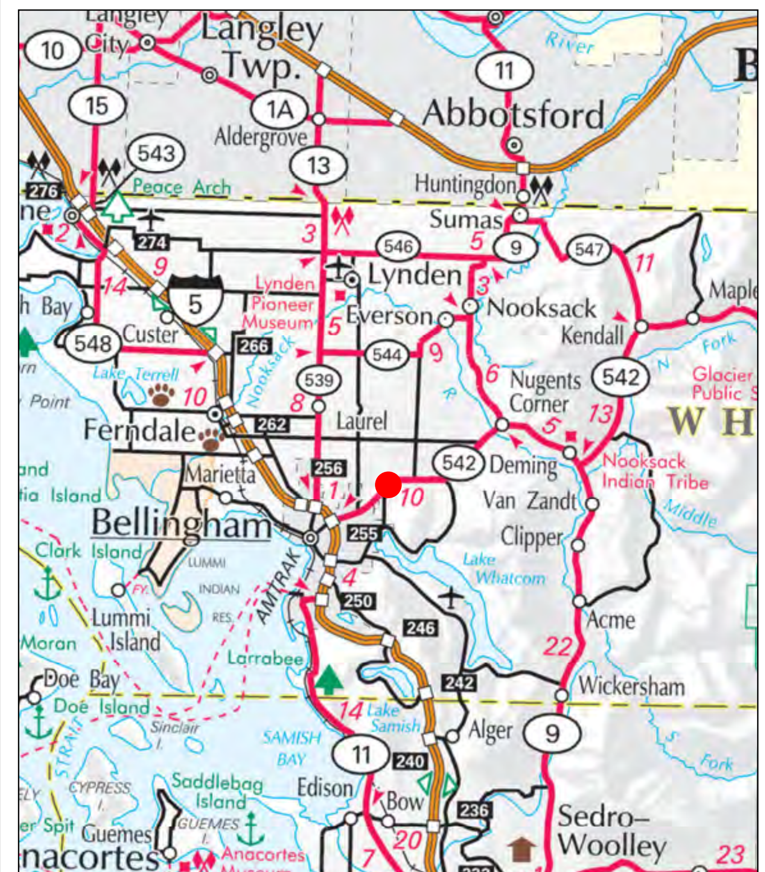
1:1,200
1 INCH = 100 FEET

FEET
0 25 50 100 150 200



1:24,000
1 INCH = 2,000 FEET

MILES
0 0.125 0.25 0.5 0.75 1



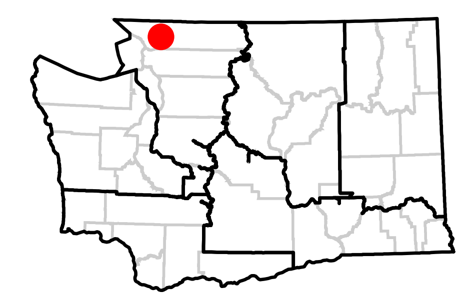
1:600,000
1 INCH = 9 MILES

MILES
0 2.5 5 10 15 20

Legend

- Site
- Milepost - 1/10th Mile
- Milepost - 1 Mile
- State Route
- NHD Rivers & Streams
- WSDOT Regions
- County Boundaries (1:500K)

N



JOB # XL6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 1: SITE VICINITY MAP

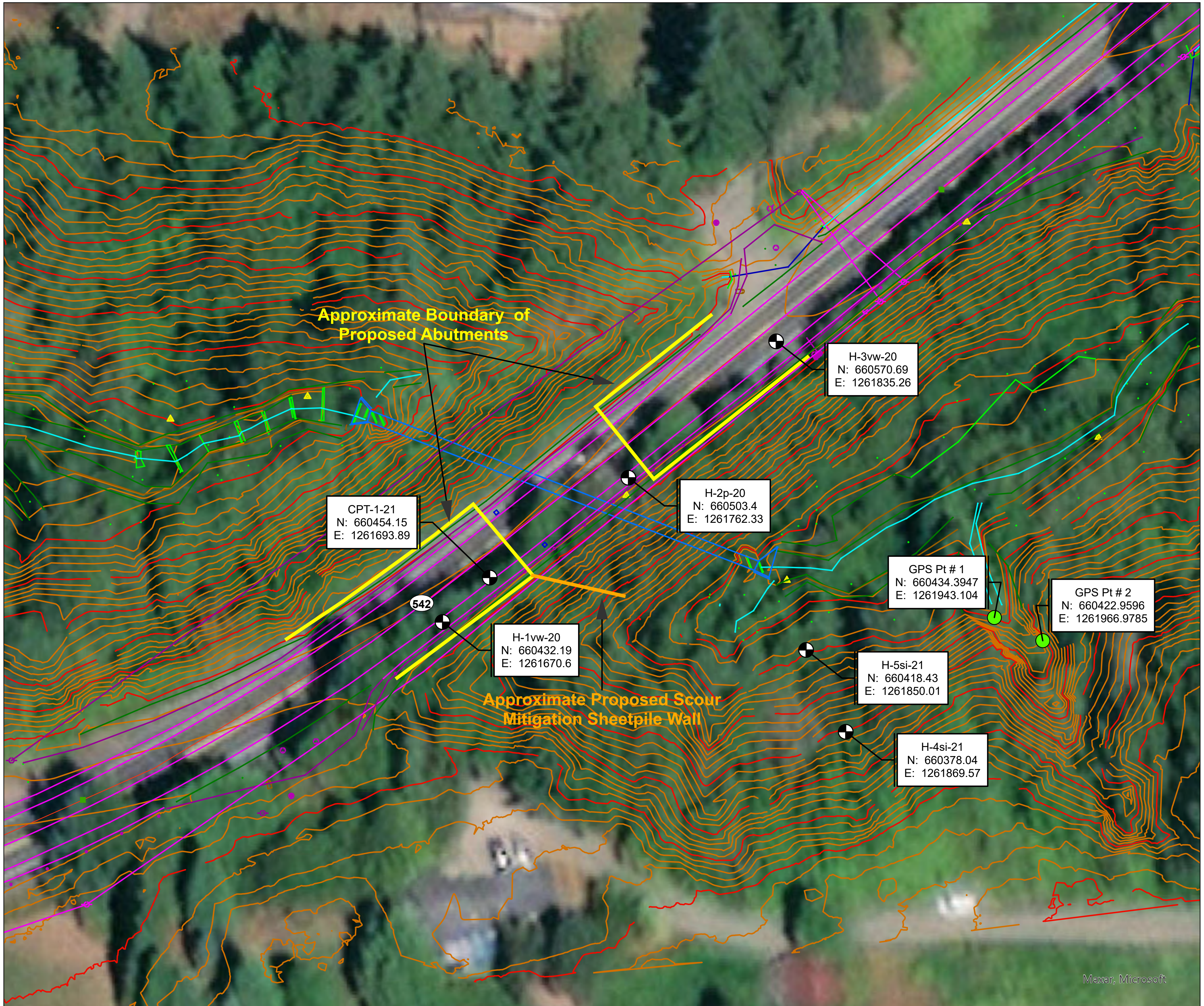
SR 542 / Squalicum Creek To Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT

Date: January 12, 2022

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-542\SR542-XL6093-JB-SqualicumCkFishPassage\Projects\SR542-XL6093-JB-SqualicumCkFishPassage.aprx



Legend

- Creek Gorge Surface Sample Location
- Test Boring Locations
- Milepost - 1/10th Mile
- Milepost - 1 Mile
- Approximate Boundary of Proposed Abutments
- Approximate Proposed Scour Mitigation Sheetpile Wall
- State Route

N

FEET

0 13 25 50 75 100

1:600

1 INCH = 50 FEET

JOB # XL6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

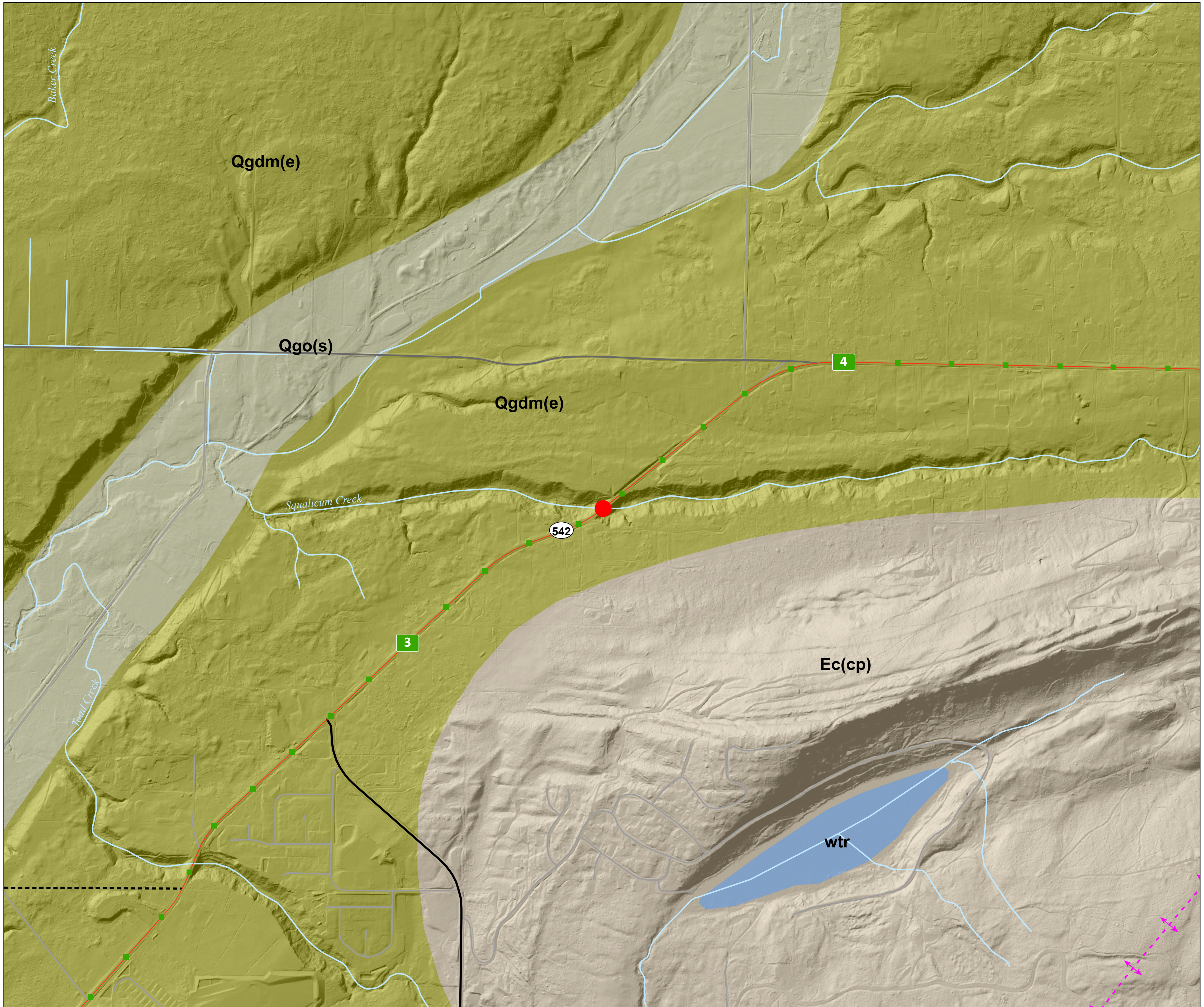
FIGURE 2: SITE EXPLORATION MAP

SR 542 / Squalicum Creek To Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: July 26, 2022

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-542\SR542-XL6093-JB-SqualicumCkFishPassage\Projects\SR542-XL6093-JB-SqualicumCkFishPassage.aprx



Legend
- Site
- Milepost - 1/10th Mile
- Milepost - 1 Mile
- State Route

Folds 100K
- Anticline - Identity and existence certain, location inferred [37]
- NHD Rivers & Streams

Geologic Units 100K

Pleistocene
- Qgdm(e) - glaciomarine drift, Fraser-age
- Qgo(s),continental glacial outwash, Fraser-age

Eocene
- Ec(cp),continental sedimentary deposits or rocks

Other
- wtr,water

100K Geology: Washington State Department of Natural Resources, Division of Geology and Earth Resources, 2010.

FEET
0 250 500 1,000 1,500 2,000
1:12,000
1 INCH = 1,000 FEET

JOB # XL6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

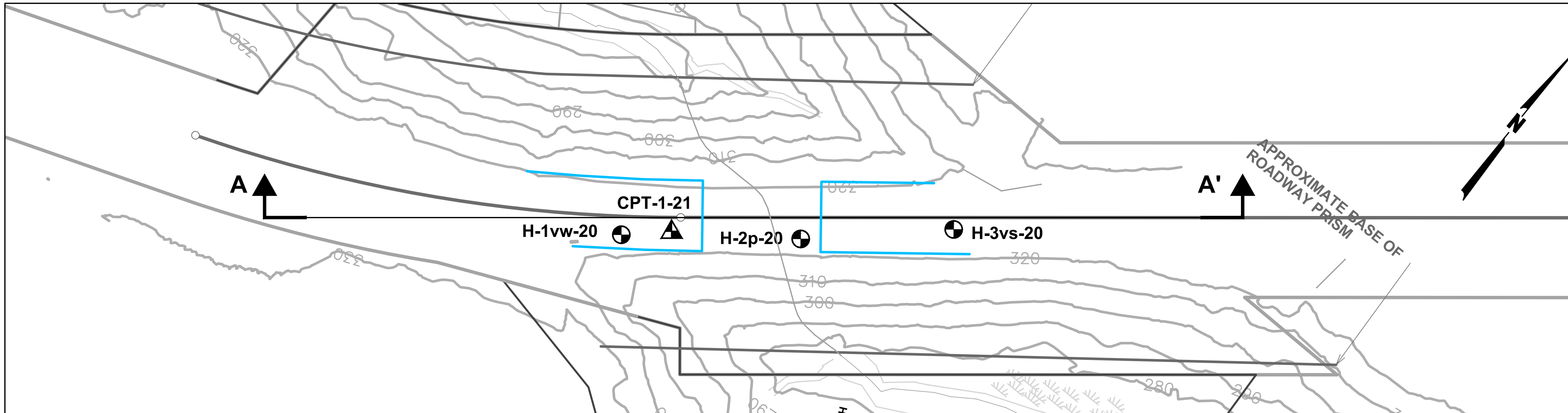
FIGURE 3: GEOLOGY MAP

SR 542 / Squalicum Creek To Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: January 12, 2022

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-542\SR542-XL6093-JB-SqualicumCkFishPassage\Projects\SR542-XL6093-JB-SqualicumCkFishPassage.aprx

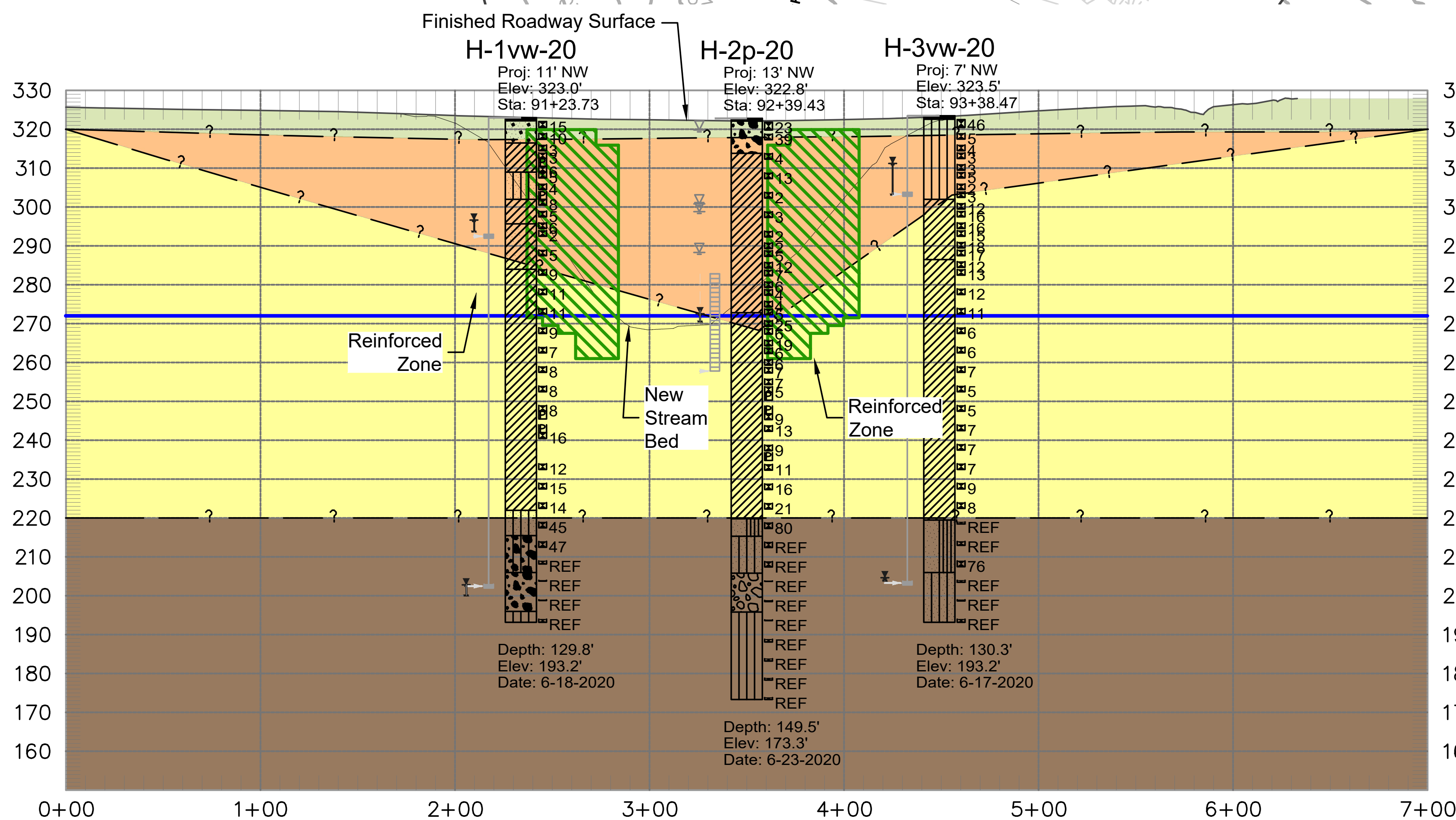


LEGEND

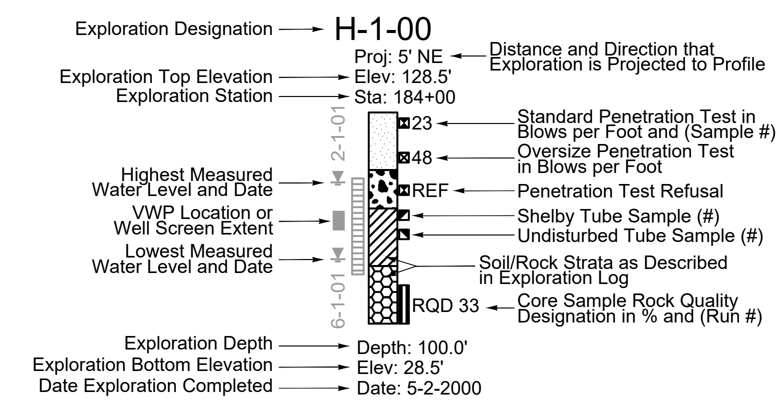
H-4si-21 Boring Designation and Approximate Location

A Generalized Subsurface Profile

CPT-1-21



EXPLORATION LOG LEGEND



ENGINEERING STRATIGRAPHIC UNIT (ESU) COLOR LEGEND

- ESU1: Fill (Coarse Grained): Loose to medium dense SAND and GRAVEL with varying amounts of silt.
- ESU2: Embankment Fill (Fine Grained): Soft to medium stiff CLAY/SILT with varying amounts of sand.
- ESU3: Bellingham Drift (Fine Grained): Medium stiff to very stiff CLAY/SILT with varying amounts of sand.
- ESU4: Bellingham Drift (Coarse Grained): Very dense SAND and GRAVEL with varying amounts of silt.
- Estimated Water Level

NOTES:

1. The datum reference for this figure is: NAD 83/91 HARN, NAVD88, SPN (ft). The exploration locations were surveyed by HQ Geotech Office.
2. ESUs, if shown, are subjective and are provided for reference only.
3. Water measurements are those measured during the monitoring period (see report text) and could be higher or lower than shown based on seasonal or other effects.
4. Stratigraphic layer divisions, if shown, are estimated at borings. Layer divisions may vary in between borings.
5. The welded wire fabric presented on this figure represents the extent of the longitudinal reinforcement only. Transverse reinforcement (i.e. perpendicular to the roadway alignment) is not shown. Transverse reinforcement should extend the full length of the exposed portions of the wall until it is embedded a minimum of 2 feet below final grade.

0 80 160

Scale in Feet

Vertical Exaggeration = 2x

JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

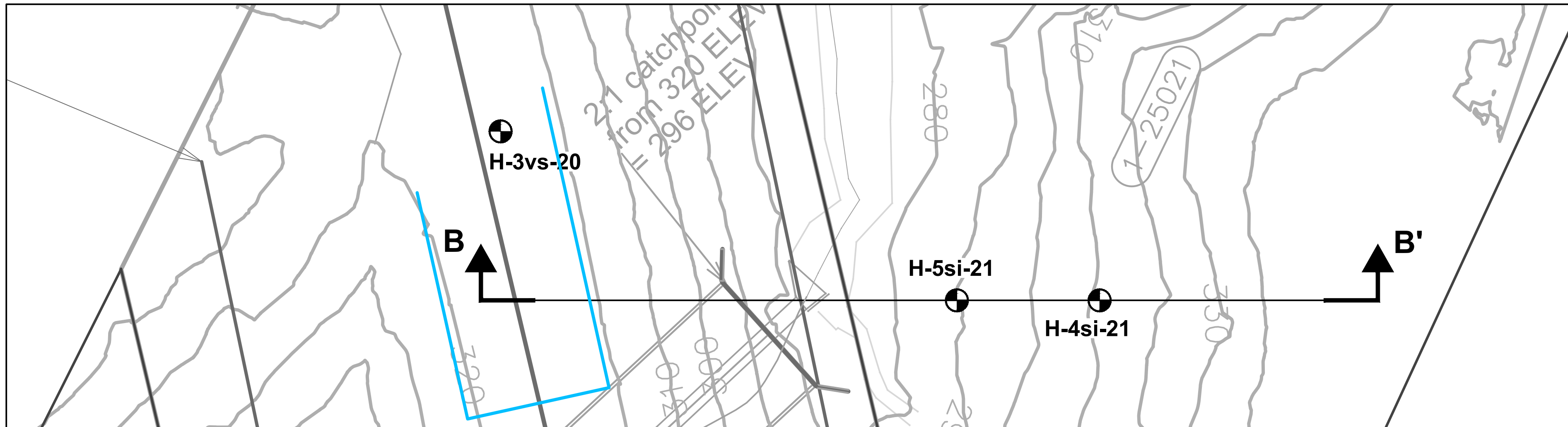
FIGURE 4: SUBSURFACE PROFILE A-A'

SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY khensen Date: July 14, 2022

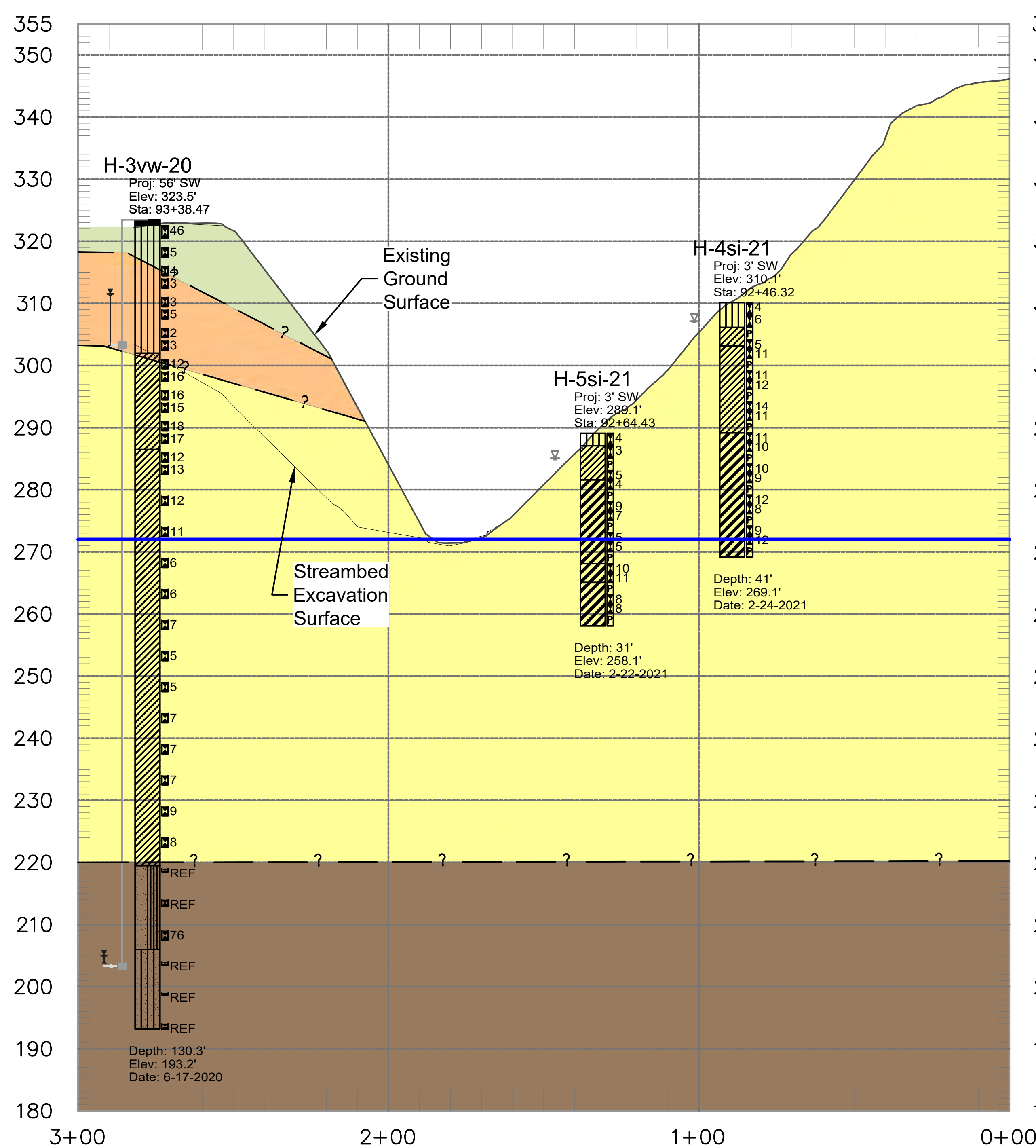
File: \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\CAD\1950127_001_SP_Xsec.dwg Layout:SectionA-Fig4



LEGEND

B-1 Boring Designation and Approximate Location

A Generalized Subsurface Profile



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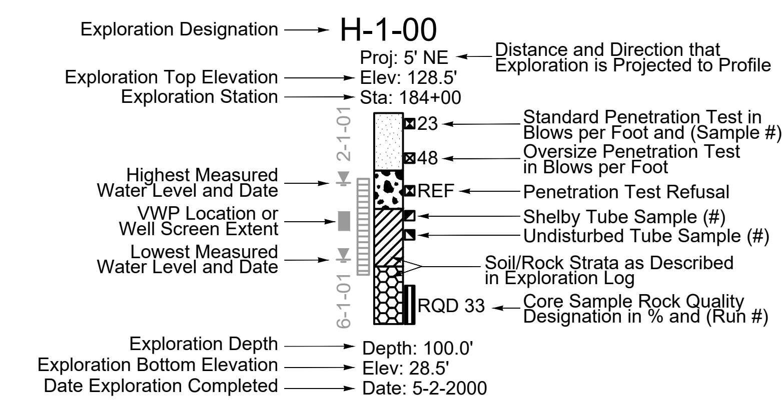


ENGINEERING STRATIGRAPHIC UNIT (ESU) COLOR LEGEND

- ESU1: Fill (Coarse Grained): Loose to medium dense SAND and GRAVEL with varying amounts of silt.
- ESU2: Embankment Fill (Fine Grained): Soft to medium stiff CLAY/SILT with varying amounts of sand.
- ESU3: Bellingham Drift (Fine Grained): Medium stiff to very stiff CLAY/SILT with varying amounts of sand.
- ESU4: Bellingham Drift (Coarse Grained): Very dense SAND and GRAVEL with varying amounts of silt.
- Estimated Water Level

NOTES:

1. The datum reference for this figure is: NAD 83/91 HARN, NAVD88, SPN (ft). The exploration locations were surveyed by HQ Geotech Office.
2. ESUs, if shown, are subjective and are provided for reference only.
3. Water measurements are those measured during the monitoring period (see report text) and could be higher or lower than shown based on seasonal or other effects.
4. Stratigraphic layer divisions, if shown, are estimated at borings. Layer divisions may vary in between borings.

0 50 100

Scale in Feet

Vertical Exaggeration = 2x

JOB# XL-6093	STATE ROUTE 542	MILEPOST(S) 3.38 to 3.52

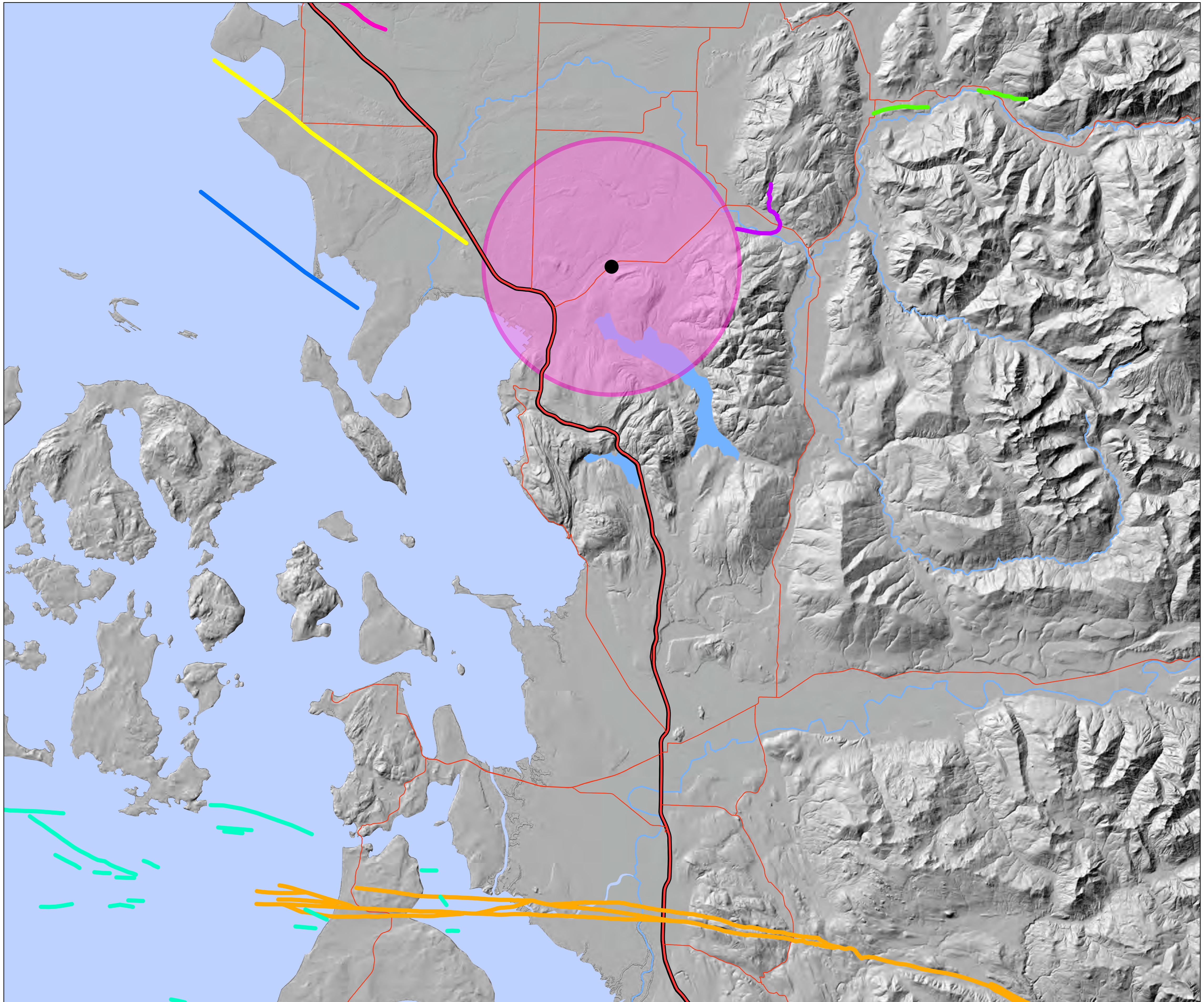
FIGURE 5: SUBSURFACE PROFILE B-B'

SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: July 14, 2022

File: C:\Users\dte\appdata\local\temp\AcPublish_1000\1950127_001_SP_Xsec.dwg Layout:SectionB-Fig5



Legend

N

- Site
- U.S. Interstate
- State Route
- Major Rivers
- Major Lakes
- Major Shorelines

USGS Quaternary Faults

- Birch Bay fault
- Boulder Creek-Canyon Creek fault
- Darrington-Devils Mountain fault
- Drayton Harbor fault scarp
- Macaulay Creek thrust
- Sandy Point fault
- unnamed faults in the Strait of Juan de Fuca and Puget Sound
- 6 Mile Buffer of Site

Fault	Miles
Macaulay Creek thrust	6.14
Birch Bay fault	6.9
Sandy Point fault	12.08
Boulder Creek-Canyon Creek fault	14.23
Drayton Harbor fault scarp	15.38
unnamed faults in the Strait of Juan de Fuca and Puget Sound	29.49
Darrington-Devils Mountain fault	29.82

MILES

0 2 4 7 11 14

1:300,000

1 INCH = 5 MILES

JOB # XL6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

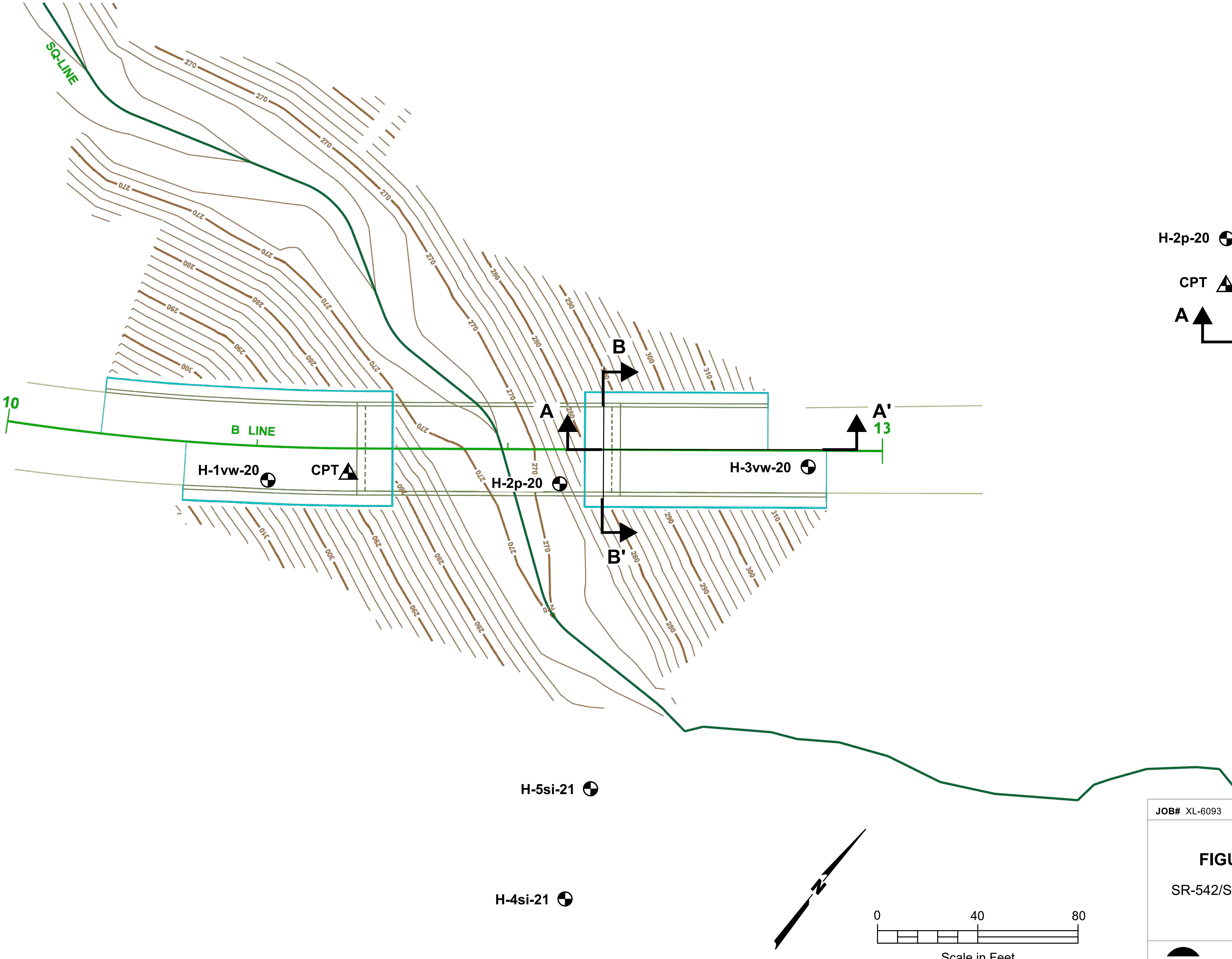
FIGURE 6: MAPPED FAULTS

SR 542 / Squalicum Creek To Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: June 9, 2022

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-542\SR542-XL6093-JB-SqualicumCkFishPassage\Projects\SR542-XL6093-JB-SqualicumCkFishPassage.aprx



LEGEND

H-2p-20 — Boring Designation and Approximate Location

CPT — Cone Penetrometer Designation and Approximate Location

A — Generalized Abutment Profile

JOB# XL-6093 **STATE ROUTE** 542 **STATION(S)** 3.38 to 3.52

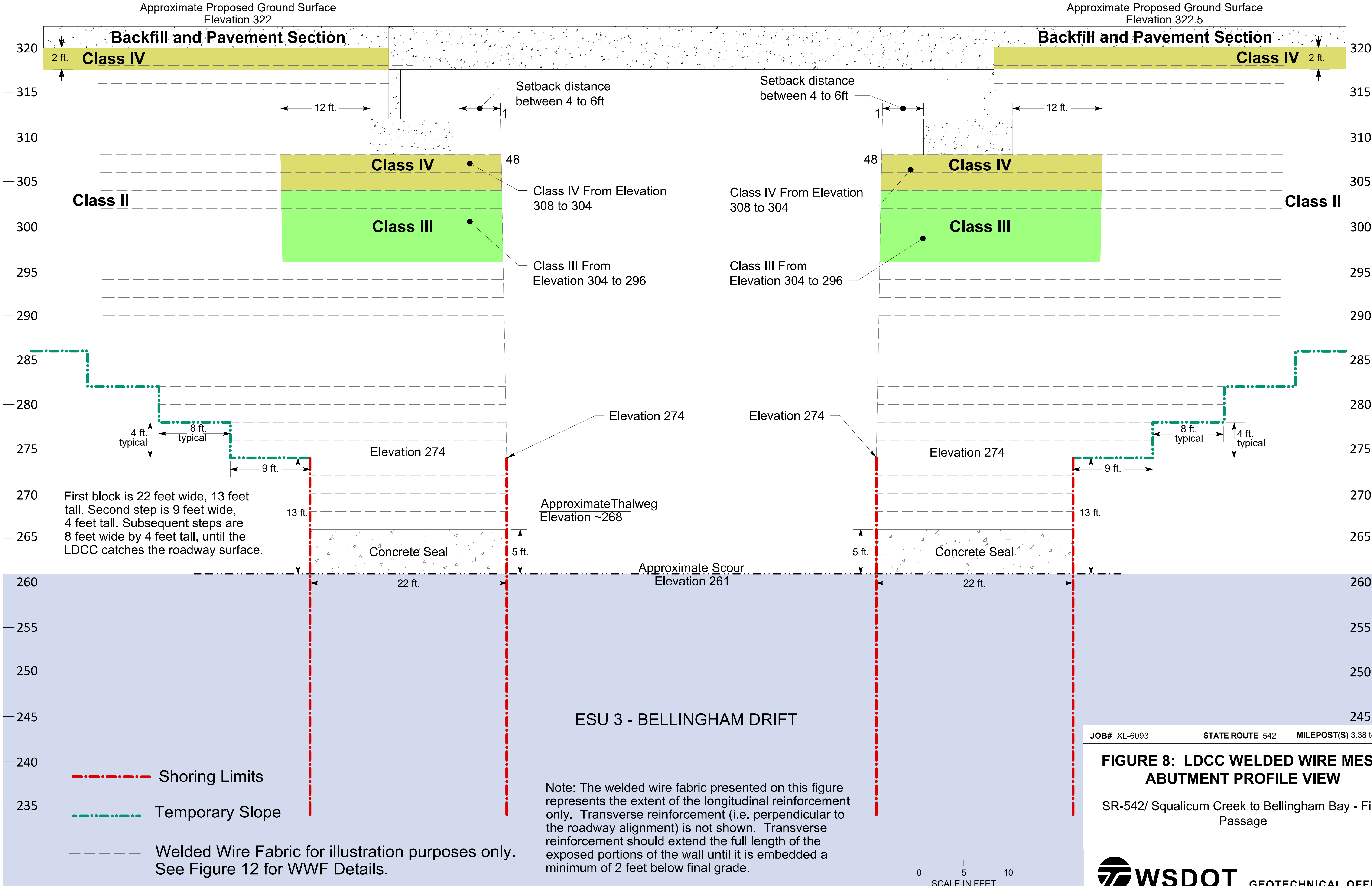
FIGURE 7: BRIDGE PLAN VIEW

SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: July 14, 2022

File: C:\Users\dte\appdata\local\temp\AcPublish_1000\1950127_001_SP_Xsec.dwg Layout:Site_Plan-Fig 7



JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

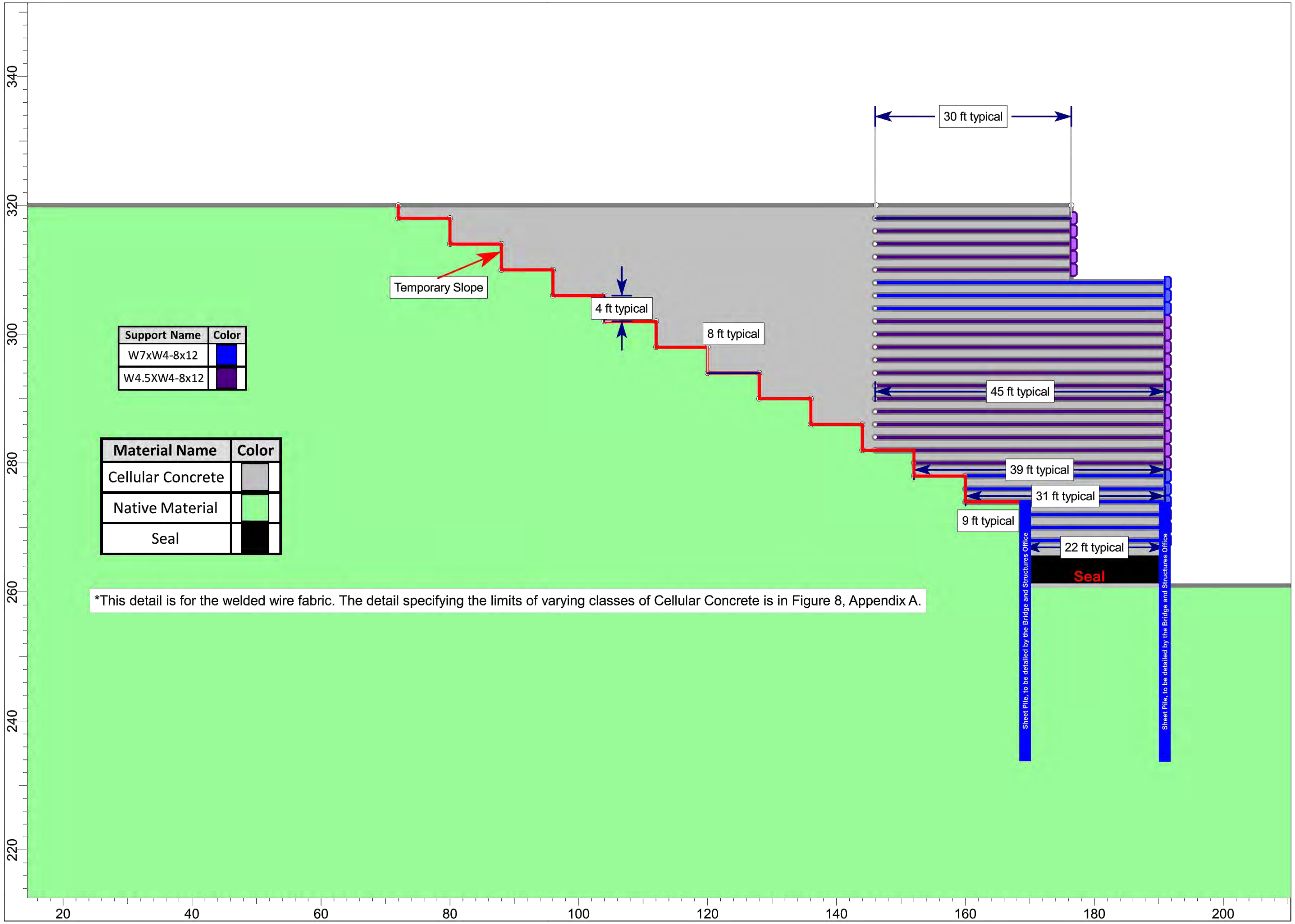
FIGURE 8: LDCC WELDED WIRE MESH ABUTMENT PROFILE VIEW

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: September 26, 2022

File: \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\CAD\1950127_001_0008_Abutment_Profile_Rev1.0.dwg Layout:Fig 8 - concrete_profile



NOTE: Design batter not shown.

Note: The welded wire fabric presented on this figure represents the extent of the longitudinal reinforcement only. Transverse reinforcement (i.e. perpendicular to the roadway alignment) is not shown. Transverse reinforcement should extend the full length of the exposed portions of the wall until it is embedded a minimum of 2 feet below final grade.

JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

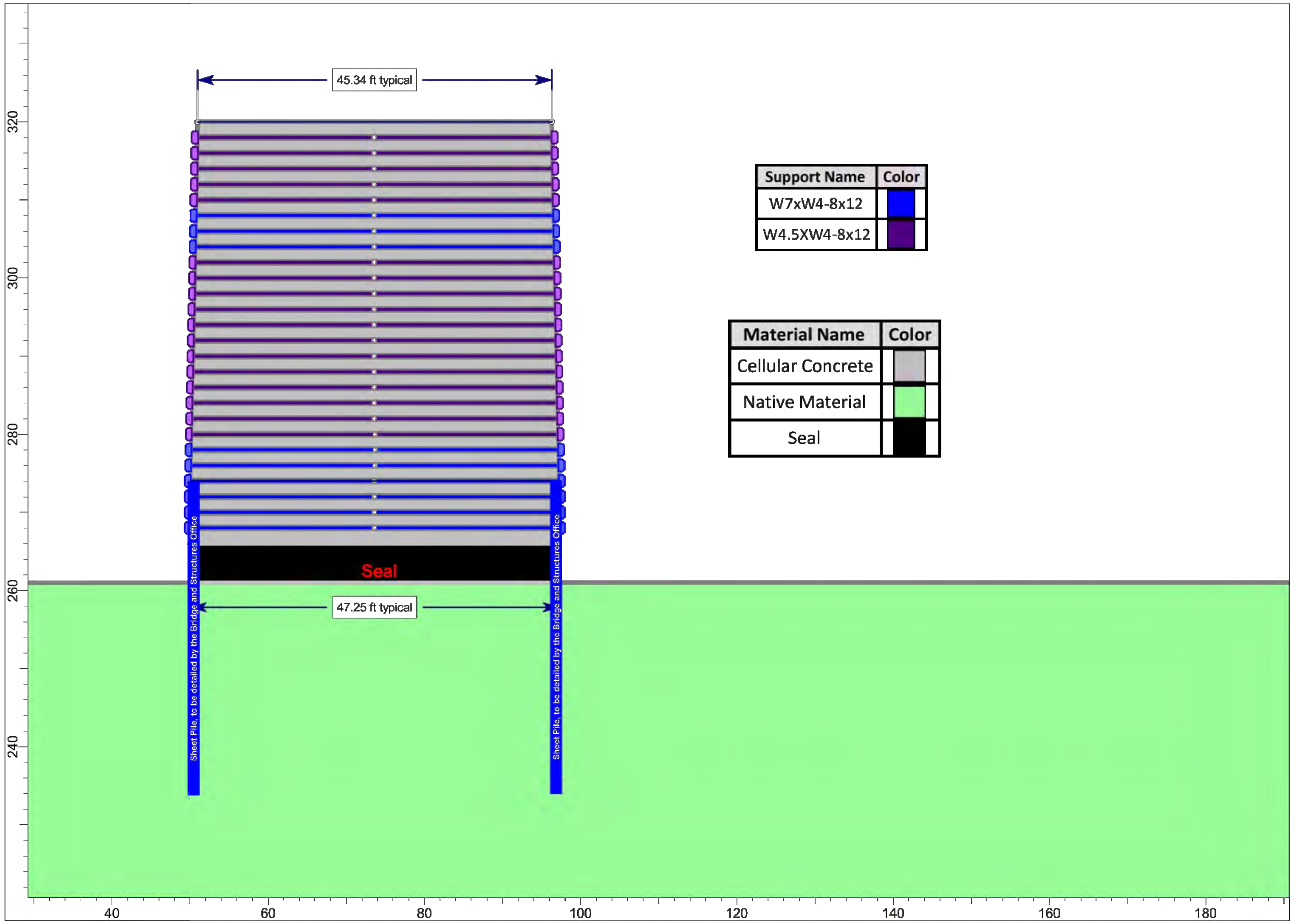
FIGURE 9:
LDCC WELDED WIRE MESH ABUTMENT: SECTION A-A'

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY mschweitzer Date: August 1, 2022

File: \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\CAD\0202868_000_0009_Section A-A_Rev3.0.dwg Layout:Section A-A'



Note: Transverse reinforcement shown in this figure should extend the full length of the exposed portions of the wall until it is embedded a minimum of 2 feet below final grade.

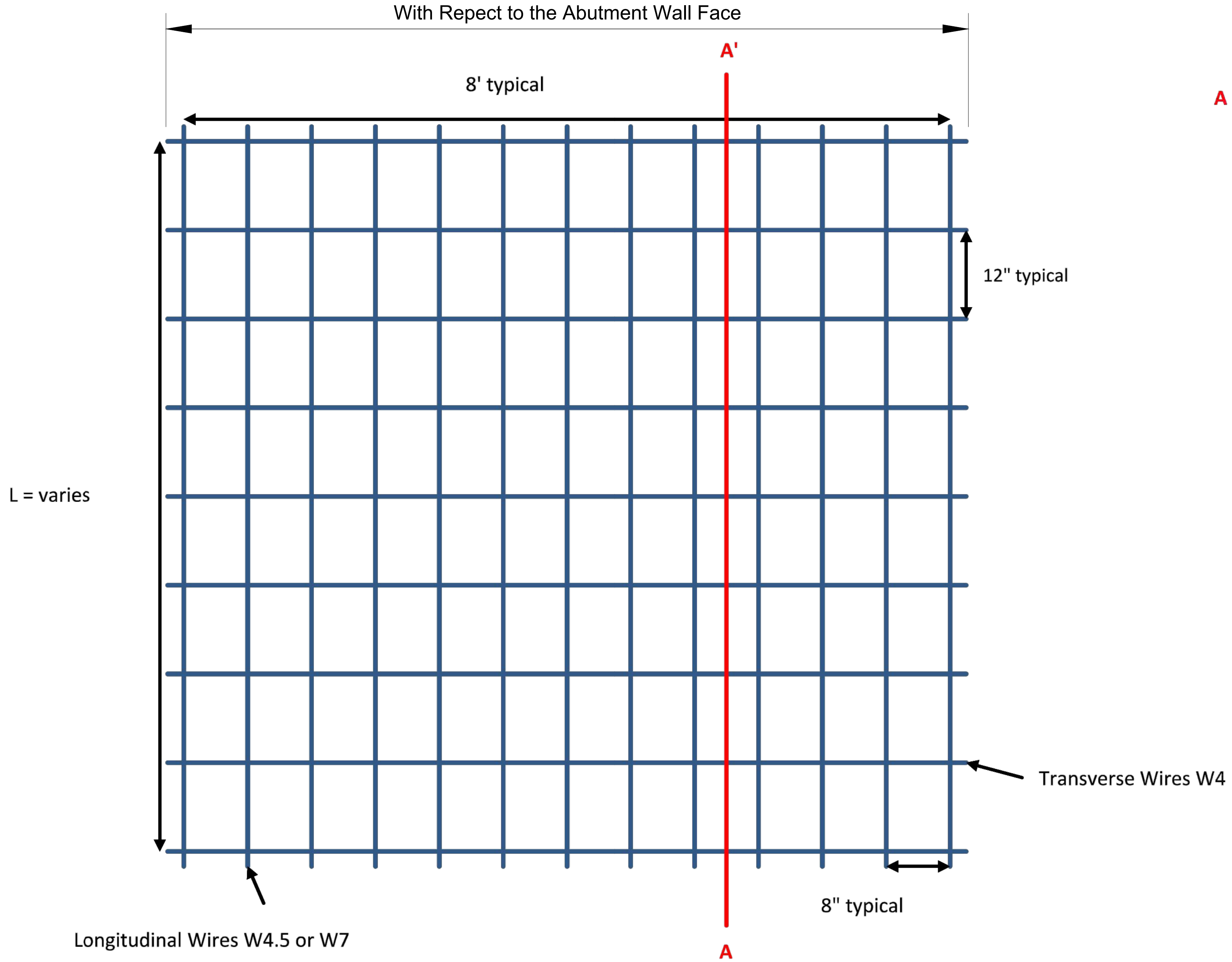
JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 10:
LDCC WELDED WIRE MESH ABUTMENT: SECTION B-B'

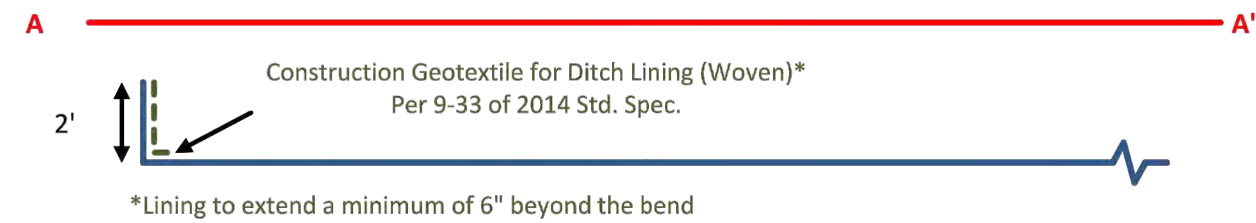
SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

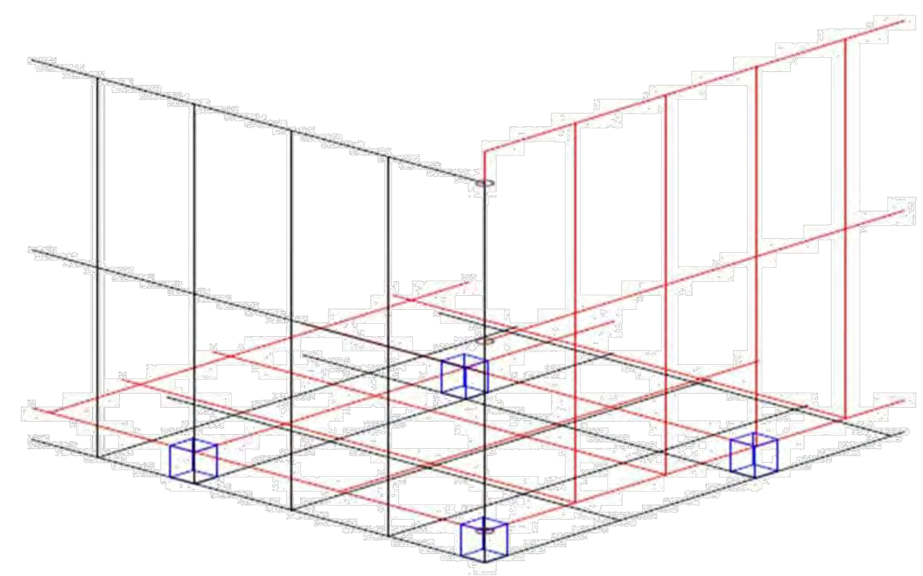
PREPARED BY mschweitzer Date: August 1, 2022



Plan View



Section View



Perspective View

- Hog Rings are to be used at edges and corners between mats
- 2" spacers are shown as blue blocks in the above drawing

JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

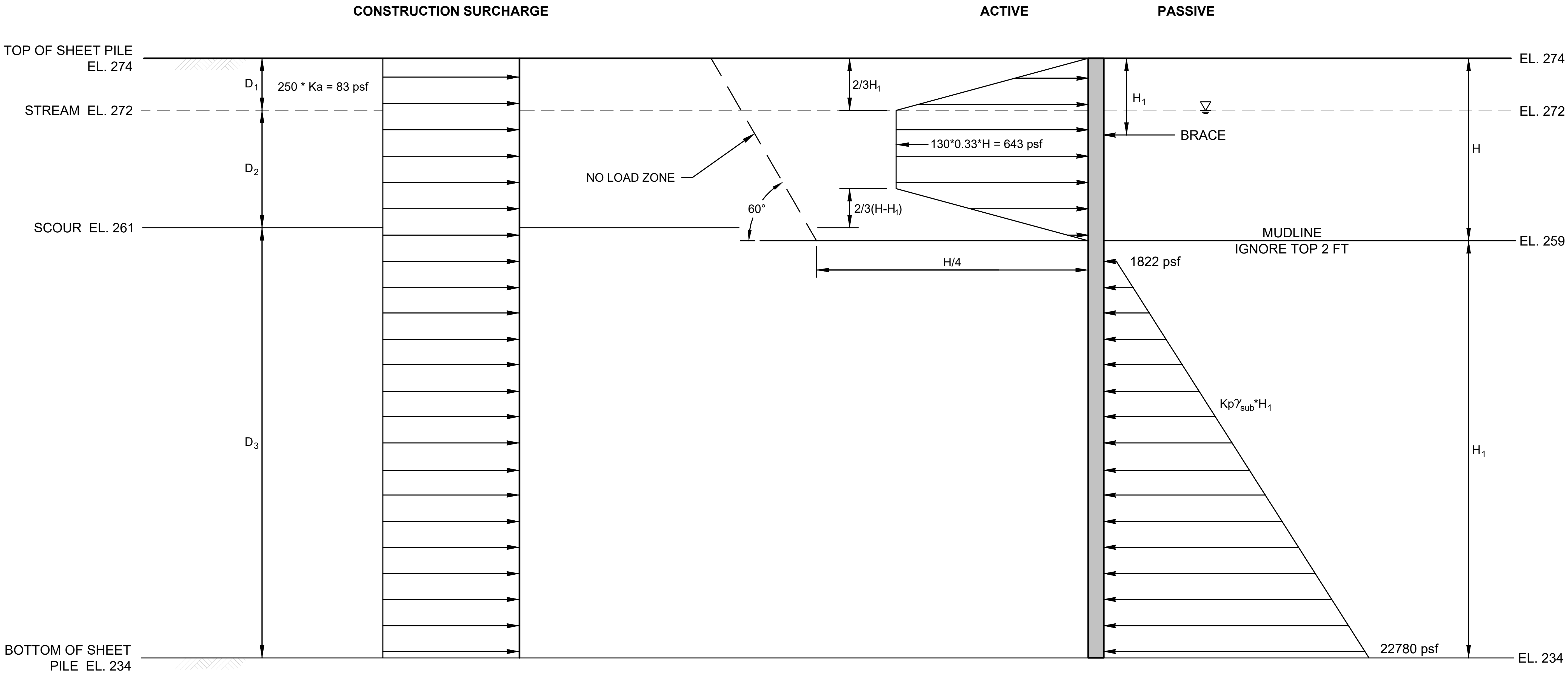
FIGURE 11: Welded Wire Fabric Detail

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: July 14, 2022

File: \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\CAD\0202868_000_0012_Welded Fabric Detail.dwg Layout:FIG 11-Detail



ESUS

γ = 130 pcf
γ_{sub} = 68 pcf
ϕ = 33 Degrees
Ka = 0.33
Kp = 13.4
Ko = 0.46

NOTES

1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF)
2. ALL PRESSURES ARE UNFACTORED. A RESISTANCE FACTOR OF 0.75 SHOULD BE APPLIED FOR THE STRENGTH LIMIT STATE PASSIVE RESISTANCE. FOR THE EXTREME LIMIT STATE THE RESISTANCE FACTOR IS 1.0. THESE FACTORS ARE FROM TABLE 11.5.7-1 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
3. THIS EARTH PRESSURE DIAGRAM ASSUMES THAT THE WALLS IN QUESTION WILL DEFORM ENOUGH TO DEVELOP THE ACTIVE EARTH PRESSURE, REQUIRING MOVEMENT OF ABOUT 2 INCHES AT THE TOP OF WALL. TO THIS END, Ka HAS BEEN USED TO CALCULATE THE ACTIVE AND CONSTRUCTION SURCHARGE PRESSURES.

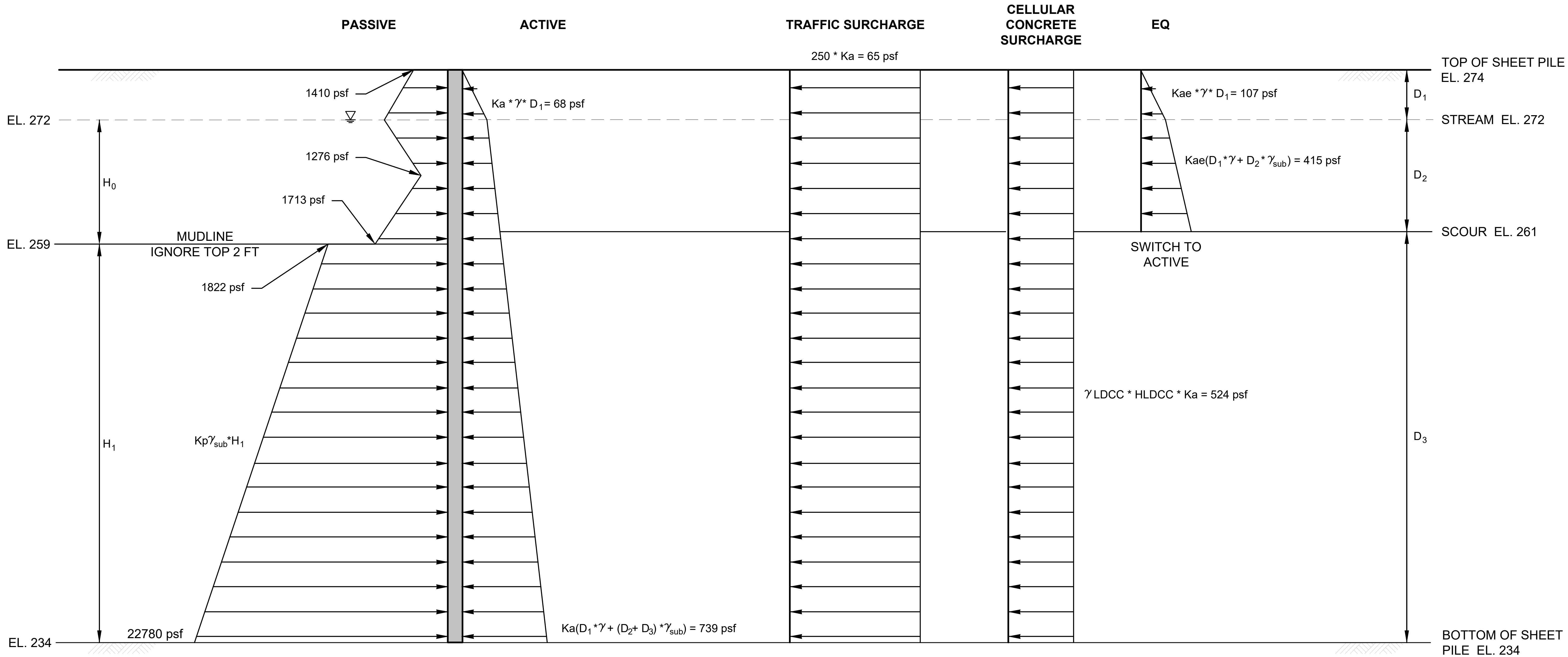
JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 12: ACTIVE CONDITION CONSTRUCTION EARTH PRESSURE DIAGRAM

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: September 26, 2022



ESUS

γ = 130 pcf
γ_{sub} = 68 pcf
ϕ = 33 Degrees
Ka = 0.26
Kp = 13.4
Ko = 0.46
Kae = 0.412

CELLULAR CONCRETE

γLDCC = 42 pcf
HLDCC = 48 ft

NOTES

1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF)
2. ALL PRESSURES ARE UNFACTORED. A RESISTANCE FACTOR OF 0.75 SHOULD BE APPLIED FOR THE STRENGTH LIMIT STATE PASSIVE RESISTANCE. FOR THE EXTREME LIMIT STATE THE RESISTANCE FACTOR IS 1.0. THESE FACTORS ARE FROM TABLE 11.5.7-1 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
3. THIS EARTH PRESSURE DIAGRAM ASSUMES THAT THE WALLS IN QUESTION WILL DEFORM ENOUGH TO DEVELOP THE ACTIVE EARTH PRESSURE, REQUIRING MOVEMENT OF ABOUT 2 INCHES AT THE TOP OF WALL. TO THIS END, Ka HAS BEEN USED TO CALCULATE THE ACTIVE AND CONSTRUCTION SURCHARGE PRESSURES.
4. THESE RECOMMENDATIONS APPLY TO THE SHEETPILE ORIENTED ON THE BACK SIDE OF THE ABUTMENT.

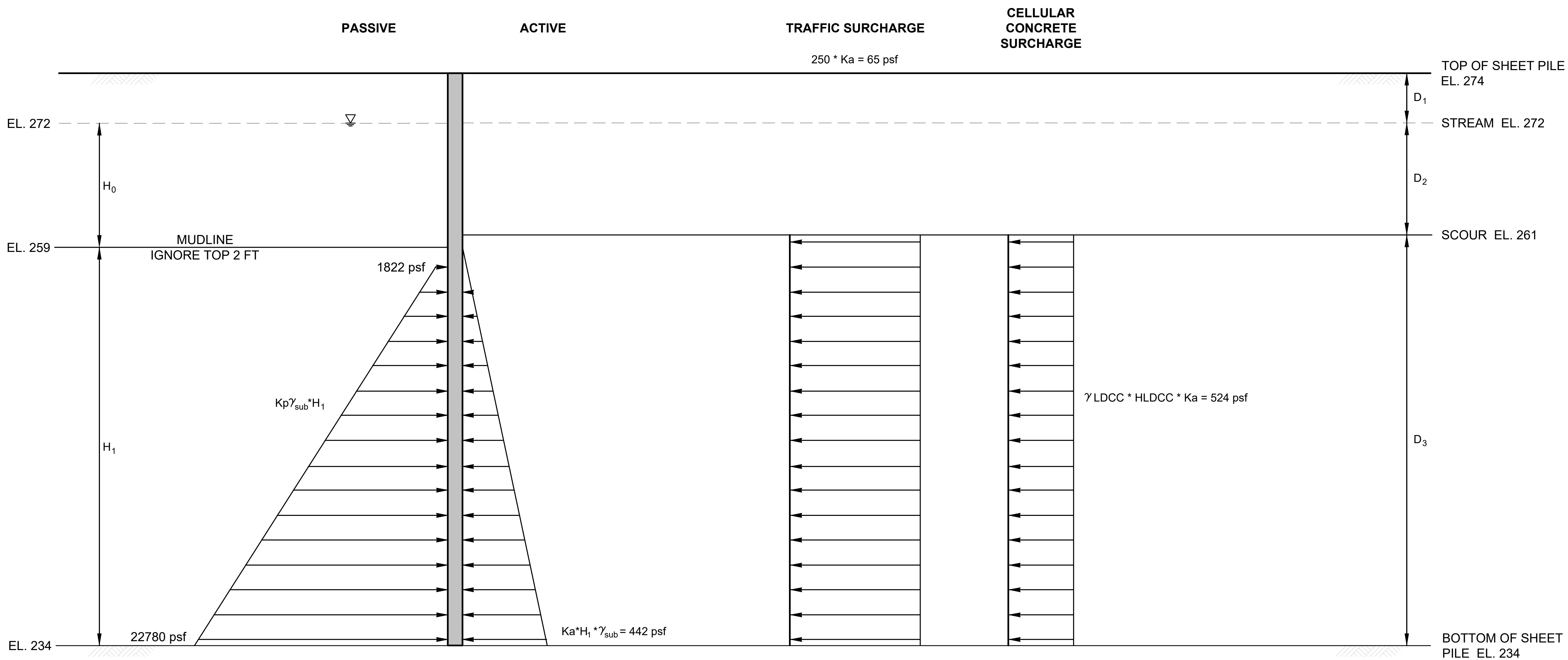
JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 13:
ACTIVE CONDITION PERMANENT EARTH PRESSURE DIAGRAM BACK SIDE SHEET PILE

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: September 26, 2022



ESUS

γ = 130 pcf
γ_{sub} = 68 pcf
ϕ = 33 Degrees
Ka = 0.26
Kp = 13.4
Ko = 0.46
Kae = 0.412

CELLULAR CONCRETE

γLDCC = 42 pcf
HLDCC = 48 ft

NOTES

1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF)
2. ALL PRESSURES ARE UNFACTORED. A RESISTANCE FACTOR OF 0.75 SHOULD BE APPLIED FOR THE STRENGTH LIMIT STATE PASSIVE RESISTANCE. FOR THE EXTREME LIMIT STATE THE RESISTANCE FACTOR IS 1.0. THESE FACTORS ARE FROM TABLE 11.5.7-1 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
3. THIS EARTH PRESSURE DIAGRAM ASSUMES THAT THE WALLS IN QUESTION WILL DEFORM ENOUGH TO DEVELOP THE ACTIVE EARTH PRESSURE, REQUIRING MOVEMENT OF ABOUT 2 INCHES AT THE TOP OF WALL. TO THIS END, Ka HAS BEEN USED TO CALCULATE THE ACTIVE AND CONSTRUCTION SURCHARGE PRESSURES.
4. THESE RECOMMENDATIONS APPLY TO THE SHEETPILE ORIENTED ON THE FRONT SIDE OF THE ABUTMENT.

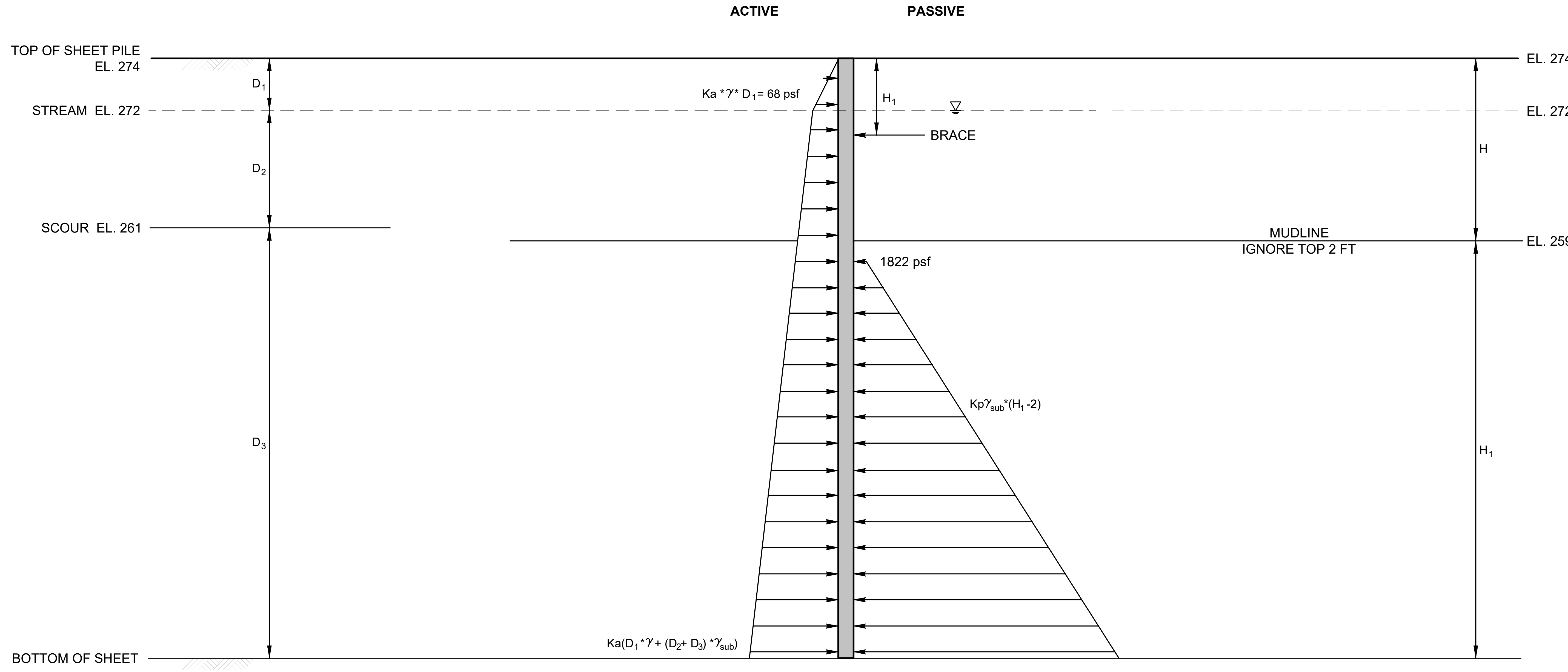
JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 14:
ACTIVE CONDITION PERMANENT EARTH PRESSURE DIAGRAM FRONT SIDE SHEET PILE

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: September 26, 2022



ESUS

γ = 130 pcf
γ_{sub} = 68 pcf
ϕ = 33 Degrees
Ka = 0.33
Kp = 13.4
Ko = 0.46

NOTES

1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF)
2. ALL PRESSURES ARE UNFACTORED. A RESISTANCE FACTOR OF 0.75 SHOULD BE APPLIED FOR THE STRENGTH LIMIT STATE PASSIVE RESISTANCE. FOR THE EXTREME LIMIT STATE THE RESISTANCE FACTOR IS 1.0. THESE FACTORS ARE FROM TABLE 11.5.7-1 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
3. THIS EARTH PRESSURE DIAGRAM ASSUMES THAT THE WALLS IN QUESTION WILL DEFORM ENOUGH TO DEVELOP THE ACTIVE EARTH PRESSURE, REQUIRING MOVEMENT OF ABOUT 2 INCHES AT THE TOP OF WALL. TO THIS END, Ka HAS BEEN USED TO CALCULATE THE ACTIVE AND CONSTRUCTION SURCHARGE PRESSURES.

JOB# XL-6093 **STATE ROUTE** 542 **MILEPOST(S)** 3.38 to 3.52

FIGURE 15:
SCOUR PROTECTION SHEETPILE WALL EARTH PRESSURE DIAGRAM

SR-542/ Squalicum Creek to Bellingham Bay - Fish Passage

WSDOT GEOTECHNICAL OFFICE

PREPARED BY DTE Date: September 26, 2022

APPENDIX A: FIELD EXPLORATIONS

CONTENTS

FIELD EXPLORATIONS

To characterize the subsurface conditions for this Project, we completed five drilled borings and one Cone Penetrometer Test (CPT) across the project site. The locations and elevations of the borings were determined by survey and are included on the boring logs in this appendix. The location coordinates on the boring logs are Washington State Plane North (WSPN) NAD83/91 coordinates. The elevations shown on the boring logs are referenced to NAVD88. The following sections describe the exploration program in more detail.

DRILLING

The drilled borings were completed by the Washington State Department of Transportation (WSDOT) Headquaters Materials Laboratory drill crews using CME 55, CME 850, and CME 45 drill rigs. The borings were completed using casing advancer drilling methods. WSDOT engineering staff supervised the field investigation effort and field exploration staff observed the exploratory drilling, collected samples, and logged the borings.

CONE PENETROMETER TESTING

One CPT was completed by ConeTec crews using C20-30Ton Truck rig. The CPT log was completed showing tip resistance and friction ratio by depth and interpreted soil classifications and strengths. Seismic cone penetration tests were also performed.

DISTURBED SAMPLING

Disturbed sampling in test borings were collected in approximate 2.5-foot intervals from the ground surface to the shallower fine-grained soil layer. The sampling interval was changed to 5-foot intervals to the bottom of the boring.

Disturbed samples were collected with a standard 2-inch outer-diameter splitspoon sampler in conjunction with Standard Penetration Tests (SPTs). In a SPT, ASTM International (ASTM) D 1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The uncorrected field SPT N-value provides a measure of *in situ* relative density of granular soils (sand and gravel), and the consistency of fine-grained or cohesive soils (Silt and Clay). Refusal blow counts were determined in general accordance with ASTM D 1586.

Field SPT N-values can be significantly affected by several factors, including the efficiency of the hammer, the type of sampler, the diameter of the borehole, the type of rod, and the length of rod between the hole (due to caving, heave, or suction), also affect the blow counts and are more difficult to quantify. For the more quantifiable variables, such as hammer efficiency, correction factors can be applied to N-values to make them more directly comparable between borings that may have been drilled to different depths or using different equipment with different energy efficiencies. The resulting corrected N-values are typically used for analysis and design. The average hammer efficiency from past efficiency calibrations performed on the rig are presented on the boring logs.

All disturbed samples were visually classified in the field, sealed to retain moisture, and returned to our laboratory for additional examination and testing.

Undisturbed Sampling

Relatively undisturbed samples were obtained using a thin-walled Shelby tube sampler in general conformance with ASTM Test Method D1587 “Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes.” The sampler is driven using the hydraulic down-pressure of the drill rig mast. Upon extraction of the undisturbed samples from the bore holes the ends were sealed with wax and caps were placed over the ends of the Shelby tubes.

Well Installations

An open standpipe piezometer well was installed in boring H-2p-20 to observe groundwater levels on a long-term basis. Details regarding well installation depths and screen interval are presented on the boring logs attached in this appendix. The wells were constructed using 1-inch-diameter PVC pipe. The annulus around the screened portion of the PVC pipe (and in some cases also above the screened interval a maximum height of 2 feet) was backfilled with a sand filter pack. The annulus above the sand filter pack was backfilled with bentonite cement grout.

Two vibrating wire piezometer (VWPs) were installed in borings H-1vwp-20 and H-3vw-20 to observed ground water levels on a long-term basis and to observe any confined aquifer conditions. Details regarding the vibrating wire piezometer installation depths are presented on the boring logs attached in this appendix. The annulus around the VWPs and the full depth of the boring was backfilled with bentonite cement grout.

Two standpipe inclinometers were installed in borings H-4si-21 and H-5si-21 to perform slope monitoring. Details regarding well installation depths and screen interval are presented on the boring logs attached in this appendix. The wells were constructed using 1-inch-diameter PVC pipe. The annulus around the inclinometers and the full depth of the boring was backfilled with bentonite cement grout.

Near the ground surface, the wells were sealed with bentonite chips in accordance with the Washington State Department of Ecology (Ecology) requirements. Where required in vegetated areas, the well was finished near the surface with open standpipe riser monuments set in concrete. The wells were constructed in accordance with Ecology regulations.

Material Descriptions

The soil samples were classified visually in the field in general accordance with Chapter 4 of the WSDOT Geotechnical Design Manual (GDM). The classification criteria in the GDM is a modified version of ASTM D 2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Once transported to the laboratory, the samples were reexamined, various laboratory tests were performed, and the field classifications were modified accordingly. We refined the visual-manual soil classifications based on the results of the laboratory tests, using the Standard Practice for Classification of Soils for Engineering Purposes (ASTM D 2487).

Boring Logs

Summary logs of the borings are attached to this appendix. A two-page explanation of the symbols and terms used on the logs is also attached just prior to the logs. Note that soil descriptions and interfaces shown on the logs are interpretive, and actual changes may be gradual.

WSDOT

LOG OF BORING H-1vw-20

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Northing: 660,432.2 feet *Latitude:* 48.799859 deg.

Driller/Inspector: Anderson, Corey (#3312T) / Henderson, Danny #2742

Easting: 1,261,670.6 feet *Longitude:* -122.404819 deg.

Start Card: RE18608 *Well Tag:* BMC-918 *Instrument:* VWP

Elevation: 323.0 feet *Collector:* Region Survey

Drilling Method: Casing Advancer *Hole Diam.:* 6 in

Horizontal/Vertical Datum: NAD 83/91 HARN, SPN / NAVD88

Equipment: CME 55 (ID:9C7-1) *Rod Type:* AWJ

Started: June 16, 2020 *Completed:* June 18, 2020

Hammer Type: AutoHammer *Historic Efficiency:* *91.1%*

Column headings: Depth (feet) | Elevation (feet) | Profile | Moisture Content (%) ✖, Fines Content (%) ○, Penetration Resistance (blows/ft) Field N ◆, SPT N_{60} ◇ (0, 20, 40, 60, 80, 100) | Blows/6" (N bpf) and other Field Data | Sample Type | Sample Number | Lab Tests | Material Description | Groundwater Data (See Note 4) | As-Built

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
0	323				*-ASPHALT*
		5 9 6 (15) Rec=0.6'	D-1		WELL-GRADED SAND WITH GRAVEL, sub-angular, medium dense, very dark grayish brown, moist, homogeneous.
	320				
5		8 7 3 (10) Rec=0.4'	D-2		WELL-GRADED GRAVEL, sub-angular, loose, very dark gray, moist, homogeneous. *-No initial recovery; used oversized sampler to recover sample.*
	315	1 2 1 (3) Rec=1.0'	D-3		SANDY SILT WITH GRAVEL, sub-rounded, very loose, brown, moist, trace wood, trace clods and/or inclusions of brown silt.
10		0 2 1 (3) Rec=1.2'	D-4	GS, AL	SANDY LEAN CLAY, very loose, dark grayish brown, wet, homogeneous, trace gravel.
		Rec=1.8'	PS-5	GS, AL, HT, SG	SANDY LEAN CLAY, sub-rounded, dark grayish brown, moist, trace gravel. *-At 12.5 ft. 100% water loss.*
	310	3 3 3 (6) Rec=0.9'	D-6	GS, AL	SANDY LEAN CLAY, loose, dark brown, moist, homogeneous, trace gravel, trace fine charcoal.
15		2 3 2 (5) Rec=0.7'	D-7		SILTY SAND, sub-angular, loose, grayish brown, moist, trace gravel, trace fine charcoal, trace clods and/or layers of dark brown organic soil.
	305	3 2 2 (4) Rec=0.7'	D-8	GS, AL	SILTY SAND, sub-angular, very loose, dark grayish brown, moist, trace gravel, trace angular clods of fine sandy silt.
20		Rec=1.6'	PS-9	GS, AL, HT, SG	SANDY LEAN CLAY, sub-rounded, dark grayish brown, moist, trace gravel.
		3 4 4 (8) Rec=1.1'	D-10	GS, AL	SANDY LEAN CLAY, loose, dark grayish brown, moist, homogeneous, trace gravel, trace organics.
	300				
25		2	D-11	GS, AL	SANDY SILT WITH GRAVEL, sub-rounded, loose, dark

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093

Route & MP Range: SR 542 MP 3.43 - 3.47

Legend: Moisture Content (%) ✖; Fines Content (%) ○; Penetration Resistance (blows/ft): Field N ◆, SPT N_{60} ◇ (scale 0, 20, 40, 60, 80, 100)

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
		3, 2, (5), Rec=1.1'	D-11	GS, AL	grayish brown, moist, FeO staining, trace clods and/or inclusions of dark brown organic soil, trace fine charcoal, chaotically mixed.
	295	3, 3, 3, (6), Rec=1.3'	D-12	GS, AL, HT, SG	SANDY LEAN CLAY, medium stiff, gray, moist, stratified, trace organics (wood), pinhole texture.
30		1, 1, 1, (2), Rec=1.2'	D-13		SANDY SILT WITH GRAVEL, sub-rounded, soft, dark greenish gray, wet, stratified, trace organics (wood). *-At 31 ft. installed HQ casing and 100% of water returned.*
	290				
35		1, 2, 3, (5), Rec=1.0'	D-14	GS, AL	CLAYEY SAND WITH GRAVEL, sub-rounded, medium stiff, dark greenish gray, wet, stratified, little to some organics (wood).
	285				
40		3, 4, 5, (9), Rec=1.0'	D-15		SANDY LEAN CLAY, stiff, gray, moist, homogeneous, trace fine gravel.
	280				
45		4, 5, 6, (11), Rec=1.5'	D-16		SANDY LEAN CLAY, stiff, gray, moist, homogeneous, trace fine gravel.
	275				*-At 49 ft. we lost about 50% of water return.*
50		3, 5, 6, (11), Rec=1.5'	D-17	GS, AL, HT, SG	SANDY LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel.
	270				
55		2, 4	D-18		SANDY LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel.

Groundwater Data: See Note 4

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
		5 (9) Rec=1.5'			
	265				
60		3 3 4 (7) Rec=1.5'	D-19	AL	SANDY LEAN CLAY, medium stiff, gray, moist, homogeneous, trace gravel.
	260				
65		2 3 5 (8) Rec=1.5'	D-20	AL	SANDY LEAN CLAY, medium stiff, gray, moist, homogeneous, trace gravel.
	255				
70		1 3 5 (8) Rec=1.5'	D-21		SANDY LEAN CLAY, medium stiff, gray, moist, homogeneous, trace gravel.
	250				*-We overreamed and installed HWT casing from 31.0 to 75.5 ft. to facilitate piston sampling.*
75		1 3 5 (8) Rec=1.5'	D-22	GS, AL	LEAN CLAY WITH SAND, medium stiff, gray, moist, homogeneous, trace gravel.
		Rec=2.0'	PS-23	GS, AL, HT, SG	LEAN CLAY WITH SAND, sub-rounded, gray, moist, trace gravel.
	245				
80		Rec=2.0'	PS-24	AL	LEAN CLAY WITH SAND, sub-rounded, gray, moist, trace gravel.
		6 8 8 (16) Rec=1.5'	D-25	GS, AL	LEAN CLAY WITH SAND, very stiff, gray, moist, homogeneous, trace gravel. *-At 82.5 ft. we reinstalled HQ casing to finish hole.*
	240				
85					

Chart: Moisture Content (%) ✖; Fines Content (%) ○; Penetration Resistance (blows/ft) Field N ◆, SPT N_{60} ◇; 0 20 40 60 80 100

Groundwater Data (See Note 4) | As-Built

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description
90	235–230	4 6 6 (12) Rec=1.5'		D-26		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
95	230–225	5 6 9 (15) Rec=1.5'		D-27		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
100	225–220	5 7 7 (14) Rec=1.5'		D-28		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
101						*-At 101 ft. drill action indicated gravels.*
105	220–215	23 23 22 (45) Rec=0.9'		D-29	GS, AL	SILTY SAND WITH GRAVEL, sub-angular, dense, gray, moist, homogeneous.
110	215–210	11 14 33 (47) Rec=1.5'		D-30		SANDY SILT WITH GRAVEL, sub-rounded, dense, gray, moist, homogeneous, diamict texture.
115	210	43 50/4" (REF) Rec=0.9'		D-31		SILTY GRAVEL WITH SAND, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.

Moisture Content (%) ✖ Fines Content (%) ○ Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ 0 20 40 60 80 100

Groundwater Data *See Note 4* As-Built

CONTINUED NEXT PAGE *(see last page for notes)*

LOG OF BORING H-1vw-20

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093

Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	SPT (N bpf)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description	Groundwater Data (See Note 4)
118.2	205	>>	50/3" (REF) Rec=0.2'	D-32		WELL GRADED GRAVEL WITH SAND, sub-rounded to sub-angular, very dense, gray, moist, homogeneous.	
120							
123.2	200	>>	50/3" (REF) Rec=0.2'	D-33		WELL GRADED GRAVEL WITH SAND, sub-rounded, very dense, gray, moist, homogeneous, trace silt. *-No initial recovery; used oversized sampler to recover sample.*	DRY
125							
128.5	195	>>	41 50/4" (REF) Rec=0.8'	D-34		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture. *-No initial recovery; used oversized sampler to recover sample.* *-A flush mount monument was installed on this boring.*	

Moisture Content (%) ✖ Fines Content (%) ○ Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ (scale 0 20 40 60 80 100)

HOLE ENDED AT 129.8 FEET ON 6-18-2020

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter(X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988. Blow counts per 6-inch increment have not been corrected.
4. The groundwater level range shown on this log represents data collected between 7/22/2020 and 6/9/2022. The blue line extends between the minimum and maximum readings collected during the monitoring period.

BAIL-RECHARGE TEST RESULTS:

Test Date: June 18, 2020
Hole Depth / Casing Depth: 45.5 feet / 44.0 feet
Water Depth Before Bailing: 12.3 feet

ELAPSED TIME (minutes)	WATER DEPTH (feet)
0	42.9
1	42.0
2	40.9
3	40.3
4	40.0
5	39.2
10	37.6
15	36.8
20	36.1
25	35.4
30	35.1
35	35.0
40	35.0

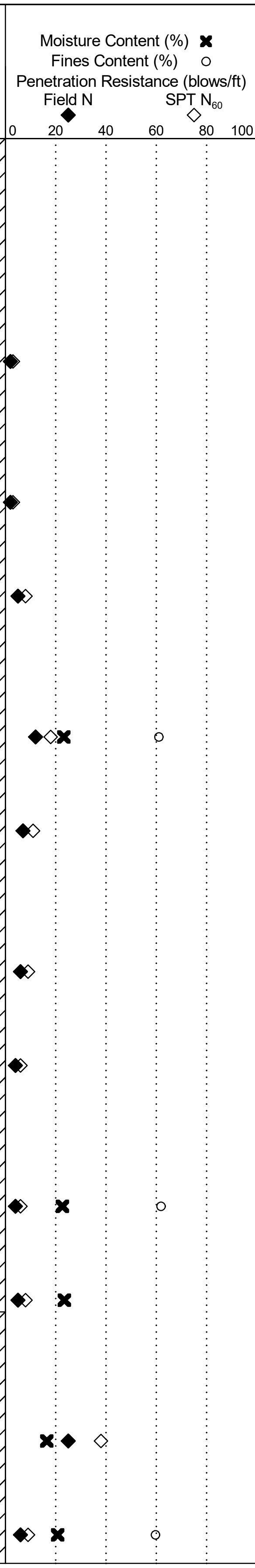
LOG OF BORING H-2p-20

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage		*Job Number:* XL6093	*Route & MP Range:* SR 542 MP 3.43 - 3.47
Northing: 660,503.4 feet	*Latitude:* 48.800059 deg.	*Driller/Inspector:* Anderson, Corey (#3312T) / Henderson, Danny #2742	
Easting: 1,261,762.3 feet	*Longitude:* -122.404445 deg.	*Start Card:* RE18608 *Well Tag:* BMC-919	*Instrument:* 1" PVC
Elevation: 322.8 feet	*Collector:* Region Survey	*Drilling Method:* Casing Advancer	*Hole Diam.:* 6 in
Horizontal/Vertical Datum: NAD 83/91 HARN, SPN / NAVD88		*Equipment:* CME 55 (ID:9C7-1)	*Rod Type:* AWJ
Started: June 18, 2020	*Completed:* June 23, 2020	*Hammer Type:* AutoHammer	*Historic Efficiency:* 91.1%

Depth (feet)	Elevation (feet)	Field N	SPT N_{60}	Moisture Content (%)	Fines Content (%)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description	Groundwater Data (See Note 4)
0									-ASPHALT	
	320					9 12 11 17 (23) Rec=0.8'	D-1		WELL GRADED GRAVEL WITH SAND, sub-rounded, medium dense, brown, wet, homogeneous.	06-19-20
5						9 28 11 (39) Rec=0.6'	D-2		WELL GRADED GRAVEL WITH SAND, sub-rounded, dense, brown, wet, trace silt, chaotically mixed.	
	315									
10						1 2 2 (4) Rec=0.4'	D-3		SANDY SILT, soft, brown, wet, homogeneous.	
	310								-At 14 ft. we lost 100% of water return.	
15						2 10 3 (13) Rec=1.2'	D-4	GS, AL	SANDY LEAN CLAY, sub-rounded, stiff, brown, wet, trace gravel, trace organics, trace clods and/or inclusions of light gray-brown silt.	
	305									
20						0 1 1 (2) Rec=1.0'	D-5		SANDY LEAN CLAY, soft, brown, wet, homogeneous, trace gravel, trace organics.	
	300									06-23-20
25						1	D-6		SANDY LEAN CLAY, soft, brown, moist, homogeneous, trace gravel, trace organics.	06-22-20

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage
Job Number: XL6093
Route & MP Range: SR 542 MP 3.43 - 3.47



Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description	Groundwater Data (See Note 4)
		1 2 (3) Rec=1.3'	D-6			
	295					
30		1 1 1 (2) Rec=1.0'	D-7		SANDY LEAN CLAY WITH GRAVEL, sub-rounded, soft, brown, moist, homogeneous, trace organics.	
	290	0 1 1 (2) Rec=0.7'	D-8		SANDY LEAN CLAY, soft, brown, moist, homogeneous, trace gravel.	
35		2 3 2 (5) Rec=0.7'	D-9		SANDY LEAN CLAY, medium stiff, brown, moist, homogeneous, trace gravel.	06-23-20
	285	1 7 5 (12) Rec=1.4'	D-10	GS, AL	SANDY LEAN CLAY, stiff, brown, moist, homogeneous, trace gravel.	
40		2 3 4 (7) Rec=1.3'	D-11		SANDY LEAN CLAY, medium stiff, brown, moist, homogeneous, trace gravel, trace organics.	
	280	2 2 4 (6) Rec=0.5'	D-12		SANDY LEAN CLAY, medium stiff, brown, moist, homogeneous, trace gravel.	
45		1 1 3 (4) Rec=0.4'	D-13		SANDY LEAN CLAY, soft, brown, moist, homogeneous, trace gravel.	
	275	1 2 2 (4) Rec=1.3'	D-14	GS, AL	SANDY LEAN CLAY, soft, brown, moist, homogeneous, trace gravel.	
50		1 2 3 (5) Rec=1.5'	D-15	AL	SANDY LEAN CLAY, medium stiff, brown, moist, stratified, trace gravel.	
	270	13 15 10 (25) Rec=1.2'	D-16	AL	SANDY LEAN CLAY WITH GRAVEL, sub-rounded, very stiff, gray and brown, moist, stratified, possible sloughed gravel.	
55		4 3	D-17	GS, AL	SANDY LEAN CLAY WITH GRAVEL, sub-rounded, medium stiff, gray, moist, homogeneous.	

CONTINUED NEXT PAGE (*see last page for notes*)

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet): 60, 65, 70, 75, 80, 85

Elevation (feet): 265, 260, 255, 250, 245, 240

Moisture Content (%) ✖ Fines Content (%) ○ Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ — 0 20 40 60 80 100

Groundwater Data — See Note 4

Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
3 (6) Rec=1.4'			
6 9 10 (19) Rec=1.0'	D-18		SANDY LEAN CLAY, very stiff, gray, moist, homogeneous, trace gravel.
1 2 4 (6) Rec=1.7'	D-19	GS, AL, HT, SG	LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous, trace gravel, trace white shell fragments.
1 2 4 (6) Rec=1.8'	D-20		LEAN CLAY WITH SAND, medium stiff, gray, moist, homogeneous, trace gravel.
1 3 4 (7) Rec=0.6'	D-21		LEAN CLAY WITH SAND, medium stiff, gray, wet, homogeneous, trace gravel.
1 4 3 (7) Rec=1.8'	D-22		LEAN CLAY WITH SAND, medium stiff, gray, moist, homogeneous, trace gravel.
1 2 3 (5) Rec=1.8'	D-23	GS, AL	LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous. *-We overreamed and installed HWT casing down to 70.5 ft. to facilitate piston sampling.*
Rec=0.0'	PS-24		NO RECOVERY
Rec=1.7'	PS-25	GS, AL, HT, SG	LEAN CLAY WITH SAND, sub-rounded, gray, wet, trace gravel.
2 4 5 (9) Rec=1.5'	D-26	GS, AL, HT, SG	SANDY LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel.
4 6 7 (13) Rec=1.5'	D-27		SANDY LEAN CLAY WITH GRAVEL, sub-rounded, stiff, gray, moist, homogeneous.
3 4 5 (9) Rec=1.5'	D-28	GS, AL	LEAN CLAY WITH SAND, stiff, gray, moist, homogeneous, trace gravel.

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093

Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description
	235	Rec=2.0'		PS-29	GS, AL, HT, SG	LEAN CLAY WITH SAND, sub-rounded, gray, moist. *-At 89 ft. we reinstalled HQ casing 100% of water returned.*
90		3 5 6 (11) Rec=1.5'		D-30	GS, AL	LEAN CLAY WITH SAND, stiff, gray, moist, homogeneous, trace gravel.
	230					
95		4 7 9 (16) Rec=1.5'		D-31		LEAN CLAY WITH SAND, very stiff, gray, moist, homogeneous, trace gravel, trace wood.
	225					
100		4 8 13 (21) Rec=1.1'		D-32		LEAN CLAY WITH SAND, very stiff, gray, moist, homogeneous, trace gravel.
	220					*-At 103.0 ft drill action indicated larger gravels.*
105		31 36 44 (80) Rec=1.0'		D-33	GS, AL	POORLY GRADED SAND WITH SILT AND GRAVEL, sub-angular, very dense, gray, moist, homogeneous.
	215					
110		17 48 50/5" (REF) Rec=1.0'		D-34		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous.
	210					
115		19 20 50/5" (REF) Rec=1.4'		D-35		SILTY SAND WITH GRAVEL, sub-angular, very dense, gray, moist, homogeneous.

Plot column: Moisture Content (%) ✖; Fines Content (%) ○; Penetration Resistance (blows/ft): Field N ◆, SPT N_{60} ◇; scale 0, 20, 40, 60, 80, 100.

Groundwater Data: See Note 4

As-Built

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage
Job Number: XL6093
Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
119.1		50/1" (REF) Rec=0.0'	D-36		NO RECOVERY *-From 119.1 to 119.7 ft., drill action indicated a cobble.*
124		50/4" (REF) Rec=0.2'	D-37		POORLY GRADED GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, fines apparently washed away.
129		50/3" (REF) Rec=0.3'	D-38		SILTY GRAVEL WITH SAND, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.
134		49 50/3" (REF) Rec=0.6'	D-39		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.
139		48 50/1" (REF) Rec=0.4'	D-40		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.
144		50/6" (REF) Rec=0.3'	D-41		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.

Depth scale: 120, 125, 130, 135, 140, 145. Elevation scale: 205, 200, 195, 190, 185, 180.

Moisture Content (%) / Fines Content (%) / Penetration Resistance (blows/ft) Field N, SPT N_{60}: 0 20 40 60 80 100

Groundwater Data: See Note 4

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Profile	Moisture Content (%) / Fines Content (%) / Penetration Resistance (blows/ft) Field N, SPT N_{60} (0 20 40 60 80 100)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description	Groundwater Data (See Note 4)	As-Built
	175		>>	50/6" (REF) Rec=0.3'		D-42		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.		

HOLE ENDED AT 149.5 FEET ON 6-23-2020

-A flush mount monument was installed on this boring.
-Robby Sheperd & Robert Walker drilled to 70.5 ft. Corey Anderson & Danny Henderson drilled to 149.5 ft.

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter(X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988. Blow counts per 6-inch increment have not been corrected.
4. The groundwater level range shown on this log represents data collected between 7/22/2020 and 6/9/2022. The blue line extends between the minimum and maximum readings collected during the monitoring period.

BAIL-RECHARGE TEST RESULTS:

Test Date: June 23, 2020
Hole Depth / Casing Depth: 35.5 feet / 33.5 feet
Water Depth Before Bailing: 22.0 feet

ELAPSED TIME (minutes)	WATER DEPTH (feet)
0	33.5
1	32.6
2	32.5
3	32.5
4	32.5
5	32.5
10	32.5

WSDOT

LOG OF BORING H-3vw-20

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Northing: 660,570.7 feet *Latitude:* 48.800247 deg.

Driller/Inspector: Walker, Robert (#2864) / Shepherd, Robert #2710

Easting: 1,261,835.3 feet *Longitude:* -122.404148 deg.

Start Card: RE-18608 *Well Tag:* BMC-920 *Instrument:* VWP

Elevation: 323.5 feet *Collector:* Region Survey

Drilling Method: Casing Advancer *Hole Diam.:* 4 in

Horizontal/Vertical Datum: NAD 83/91 HARN, SPN / NAVD88

Equipment: CME 850 (ID:9C2-5) *Rod Type:* AWJ

Started: June 16, 2020 *Completed:* June 17, 2020

Hammer Type: AutoHammer *Historic Efficiency:* *81.3%*

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
					-ASPHALT
		15 21 25 12 (46) Rec=1.5'	D-1		SILTY SAND WITH GRAVEL, sub-rounded, dense, light brown, moist, homogeneous.
	320				
5		3 2 3 (5) Rec=0.5'	D-2		SILTY SAND WITH GRAVEL, sub-rounded, medium stiff, brown, moist, homogeneous, trace organics.
	315	2 2 2 (4) Rec=1.2'	D-3		SANDY SILT, sub-rounded, soft, brown, moist, trace gravel, trace organics (wood, roots), chaotically mixed.
10		1 1 2 (3) Rec=1.1'	D-4		SANDY SILT, sub-rounded, soft, brown, moist, trace gravel, trace organics (wood, roots), chaotically mixed, trace angular clods of gray-brown silt in brown matrix.
	310	2 1 2 (3) Rec=0.9'	D-5		SANDY SILT, sub-rounded, soft, gray, wet, homogeneous, trace gravel, few to little wood fragments.
15		1 2 3 (5) Rec=1.0'	D-6		SANDY SILT, sub-rounded, medium stiff, gray, wet, homogeneous, trace gravel, few to little wood fragments.
	305	1 1 1 (2) Rec=1.5'	D-7	GS, AL, LOI	SANDY SILT, sub-rounded, very loose, gray, wet, homogeneous, trace gravel, trace to few wood and roots.
20		0 2 1 (3) Rec=1.6'	D-8		SANDY SILT, sub-rounded, very loose, light brownish gray to grayish brown, wet, homogeneous, trace gravel, trace to few wood and roots.
					-Drilling stiffened up at approximately 21.5 ft.
	300	2 5 7 (12) Rec=0.9'	D-9	GS, AL	SANDY LEAN CLAY, sub-angular, stiff, light brownish gray to grayish brown, moist, homogeneous, trace wood fragments and fibrous roots, FeO stains throughout.
25			D-10		

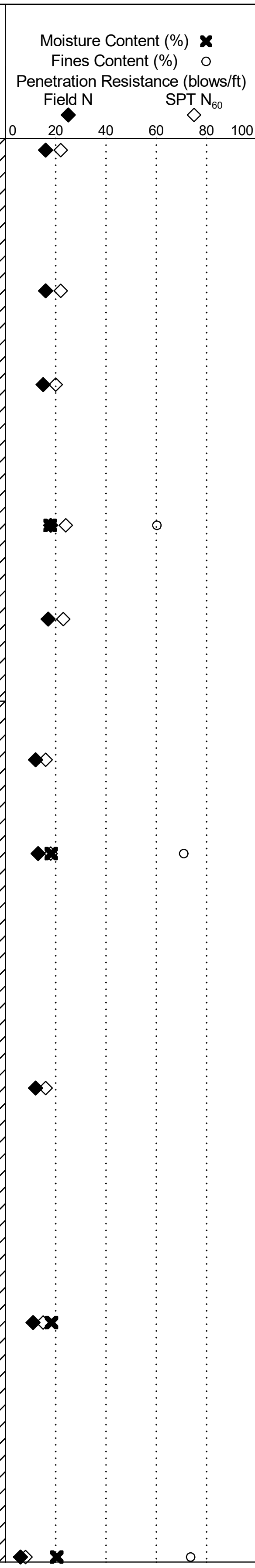
Moisture Content (%) ✖ Fines Content (%) ○ Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ (0 20 40 60 80 100)

Groundwater Data: DRY (See Note 4)

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage
Job Number: XL6093
Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
		3, 6, 10, (16), Rec=0.6'	D-10		SANDY LEAN CLAY, very stiff, brown, wet, homogeneous, mostly fine to medium sand, trace coarse sand. *-Starting losing approximately 85% of fluid return at approximately 27.5 ft.*
	295	4, 7, 9, (16), Rec=1.5'	D-11		SANDY LEAN CLAY, sub-rounded, very stiff, brown, moist, homogeneous, trace gravel, mostly fine to medium sand, trace coarse sand.
30		4, 6, 9, (15), Rec=1.7'	D-12		SANDY LEAN CLAY, sub-rounded, stiff, brown, moist, homogeneous, trace gravel, mostly fine to medium sand, trace coarse sand.
	290	5, 8, 10, (18), Rec=2.2'	D-13	GS, AL, HT, SG	SANDY LEAN CLAY, sub-rounded, very stiff, brown, moist, trace gravel, mostly fine to medium sand, trace coarse sand, trace streaks of gray.
35		4, 7, 10, (17), Rec=1.8'	D-14		SANDY LEAN CLAY, sub-rounded, very stiff, brown, moist, trace gravel, mostly fine to medium sand, trace coarse sand, trace streaks of gray.
	285	3, 5, 7, (12), Rec=1.9'	D-15		LEAN CLAY WITH SAND, sub-rounded, stiff, gray, moist, homogeneous, trace gravel.
40		3, 5, 8, (13), Rec=1.8'	D-16	GS, AL	LEAN CLAY WITH SAND, sub-rounded, stiff, gray, moist, homogeneous, trace gravel.
	280				
45		2, 4, 8, (12), Rec=1.8'	D-17		LEAN CLAY WITH SAND, sub-rounded, stiff, gray, moist, homogeneous, trace gravel.
	275				
50		3, 4, 7, (11), Rec=1.8'	D-18	AL	LEAN CLAY WITH SAND, sub-rounded, stiff, gray, moist, homogeneous, trace gravel.
	270				
55		1	D-19	GS, AL	LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous, trace fine to coarse gravel.



Groundwater Data: See Note 4

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
		3 3 (6) Rec=1.8'	D-19	GS, AL	
	265				
60		1 3 3 (6) Rec=1.1'	D-20		LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous, trace fine to coarse gravel.
	260				
65		2 3 4 (7) Rec=0.8'	D-21		LEAN CLAY WITH SAND, medium stiff, gray, moist, homogeneous, primarily fine to medium sand, trace coarse sand.
	255				
70		0 2 3 (5) Rec=1.8'	D-22	GS, AL, HT, SG	LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous.
	250				
75		1 2 3 (5) Rec=1.8'	D-23		LEAN CLAY WITH SAND, sub-angular, medium stiff, gray, moist, homogeneous.
	245				
80		1 3 4 (7) Rec=0.8'	D-24		LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous.
	240				
85		0 2	D-25		LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous.

Column headers: Profile; Moisture Content (%) ✖; Fines Content (%) ○; Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ (0 20 40 60 80 100); Sample Type; Groundwater Data (See Note 4); As-Built

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093

Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
		5 (7) Rec=1.8'			
	235				
90		0 3 4 (7) Rec=1.8'	D-26	GS, AL	LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous, trace gravel.
	230				
95		1 3 6 (9) Rec=1.8'	D-27		LEAN CLAY WITH SAND, sub-rounded, stiff, gray, moist, homogeneous, trace fine white shell fragments.
	225				
100		2 3 5 (8) Rec=1.8'	D-28		LEAN CLAY WITH SAND, sub-rounded, medium stiff, gray, moist, homogeneous.
	220				
					-Drill action indicates gravel at approximately 104 ft.
105		50/6" (REF) Rec=0.9'	D-29		WELL GRADED GRAVEL WITH SAND, sub-rounded, very dense, light gray, wet, homogeneous.
					-Drill action indicates less gravel at approximately 107 ft.
	215				
110		25 50/6" (REF) Rec=0.8'	D-30		POORLY GRADED SAND, sub-rounded, very dense, gray, wet, homogeneous, trace to few gravel.
	210				
115		21 26 50 (76)	D-31	GS, AL	POORLY GRADED SAND WITH SILT, sub-rounded, very dense, gray, wet, homogeneous, trace gravel.

Chart legend: Moisture Content (%) ✖; Fines Content (%) ○; Penetration Resistance (blows/ft) Field N ◆, SPT N_{60} ◇; scale 0 20 40 60 80 100. Groundwater Data: See Note 4. As-Built.

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description
		Rec=0.7'				
	205					
120		50/6" (REF) Rec=0.6'		D-32		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.
	200					
125		50/3" (REF) Rec=0.6'		D-33		SILTY GRAVEL WITH SAND, sub-angular, very dense, gray, moist, homogeneous, diamict texture.
	195					
130		22 50/3" (REF) Rec=0.9'		D-34		SANDY SILT WITH GRAVEL, sub-rounded, very dense, gray, moist, homogeneous, diamict texture.

Column headings: Moisture Content (%); Fines Content (%); Penetration Resistance (blows/ft) Field N, SPT N_{60} (scale 0, 20, 40, 60, 80, 100); Profile; Groundwater Data (See Note 4); As-Built.

HOLE ENDED AT 130.3 FEET ON 6-17-2020

-No monument was installed on this boring.

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter(X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988. Blow counts per 6-inch increment have not been corrected.
4. The groundwater level range shown on this log represents data collected between 7/22/2020 and 6/9/2022. The blue line extends between the minimum and maximum readings collected during the monitoring period.

BAIL-RECHARGE TEST RESULTS:

Test Date: June 17, 2020
Hole Depth / Casing Depth: 26.0 feet / 24.0 feet
Water Depth Before Bailing: 4.6 feet

ELAPSED TIME (minutes)	WATER DEPTH (feet)
0	21.4
1	18.8
2	16.7
3	16.7
4	15.9
5	15.5
10	14.2
15	13.7
20	13.5
25	13.3

LOG OF BORING H-4si-21

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage
Job Number: XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47
Northing: 660,378.0 feet *Latitude:* 48.799721 deg.
Driller/Inspector: Wilson, Jamie (#2941) / Henderson, Danny (#2742)
Easting: 1,261,869.6 feet *Longitude:* -122.403990 deg.
Start Card: RE18577 *Well Tag:* BMC-916 *Instrument:* INC
Elevation: 310.1 feet *Collector:* Region Survey
Drilling Method: Casing Advancer *Hole Diam.:* 6 in
Horizontal/Vertical Datum: NAD 83/91 HARN, SPN / NAVD88
Equipment: CME 45 (ID:9C4-4) *Rod Type:* HWT
Started: February 23, 2021 *Completed:* February 24, 2021
Hammer Type: Autohammer *Historic Efficiency:* 88.3%

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description
		0 2 2 3 (4) Rec=0.5'	D	D-1		SILT, soft, dark grayish brown, wet, homogeneous, few organics (roots, stems, wood, fibers).
		3 3 3 2 (6) Rec=0.0'	D	D-2		NO RECOVERY
5	305	Rec=1.7'	P	PS-3		LEAN CLAY WITH GRAVEL, subrounded, dark grayish brown, moist.
		2 2 3 (5) Rec=1.1'	D	D-4		LEAN CLAY, medium stiff, dark grayish brown gray, moist, homogeneous, trace to few fine gravel, trace sand.
		3 4 7 (11) Rec=1.3'	D	D-5		LEAN CLAY, subrounded, stiff, gray, moist, homogeneous, trace to few fine gravel, trace sand.
10	300	Rec=1.8'	P	PS-6	AL	LEAN CLAY, gray, moist, trace gravel.
		3 4 7 (11) Rec=1.5'	D	D-7		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
		3 5 7 (12) Rec=1.5'	D	D-8		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand, trace white bivalve shells.
15	295	Rec=1.4'	P	PS-9		LEAN CLAY, gray, moist, trace gravel.
		4 5 9 (14) Rec=1.5'	D	D-10		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
		3 5 6 (11) Rec=1.5'	D	D-11		LEAN CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
20	290	Rec=1.7'	P	PS-12		LEAN CLAY, gray, moist.
		3 5 6 (11) Rec=1.5'	D	D-13		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
		3 4 6 (10) Rec=1.5'	D	D-14		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.
25		Rec=1.9'	P	PS-15		FAT CLAY, gray, moist.

Water Observations: 02-23-21 ATD

Moisture Content (%) ✖ Fines Content (%) ○ Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ (0 20 40 60 80 100)

Columns: Profile, Water Observations, As-Built

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage *Job Number:* XL6093 *Route & MP Range:* SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Profile	Moisture Content (%) ✖ / Fines Content (%) ○ / Penetration Resistance (blows/ft) Field N ◆ SPT N_{60} ◇ (0 20 40 60 80 100)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description	Water Observations	As-Built
				Rec=1.9'		PS-15		FAT CLAY, gray, moist.		
				3 5 5 (10) Rec=1.5'		D-16		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.		
				3 4 5 (9) Rec=1.5'		D-17		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand, trace white shell fragments.		
30	280			Rec=2.0'		PS-18		FAT CLAY, gray, moist.		
				4 6 6 (12) Rec=1.5'		D-19		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel.		
				3 4 4 (8) Rec=1.5'		D-20		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel.		
35	275			Rec=2.0'		PS-21		FAT CLAY, gray, moist, trace gravel.		
				3 4 5 (9) Rec=1.5'		D-22		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel.		
				4 5 7 (12) Rec=0.0'		D-23		NO RECOVERY		
40	270			Rec=2.0'		PS-24		FAT CLAY, gray, moist, trace gravel.		

HOLE ENDED AT 41.0 FEET ON 2-24-2021

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter(X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988. Blow counts per 6-inch increment have not been corrected.
4. The groundwater level(s), if shown, represents observations made during drilling and/or stabilized water measured during a bail test. The groundwater level should be considered approximate and will vary based on seasonal and other effects.

BAIL-RECHARGE TEST RESULTS:

Test Date: February 23, 2021
Hole Depth / Casing Depth: 39.0 feet / 41.0 feet
Water Depth Before Bailing: 3.0 feet
Note: will check water again in the morning before installing slope

ELAPSED TIME (minutes)	WATER DEPTH (feet)
0	38.7
1	37.7
2	37.6
3	37.4
4	37.3
5	37.2
10	36.9
15	36.9
20	36.9

WSDOT

Project:	SR-542/Squalicum Creek to Bellingham Bay - Fish Passage	*Job Number:*	XL6093
Route & MP Range:	SR 542 MP 3.43 - 3.47	*Driller/Inspector:*	Wilson, Jamie (#2941) / Henderson, Danny (#2742)
Northing:	660,418.4 feet	*Latitude:*	48.799831 deg.
Easting:	1,261,850.0 feet	*Longitude:*	-122.404074 deg.
Start Card:	RE18577	*Well Tag:*	BMC-917
Instrument:	INC	*Elevation:*	289.1 feet
Collector:	Region Survey	*Drilling Method:*	Casing Advancer
Hole Diam.:	6 in	*Horizontal/Vertical Datum:*	NAD 83/91 HARN, SPN / NAVD88
Equipment:	CME 45 (ID:9C4-4)	*Rod Type:*	HWT
Started:	February 21, 2021	*Completed:*	February 22, 2021
Hammer Type:	Autohammer	*Historic Efficiency:*	88.3%

Columns: Depth (feet) | Elevation (feet) | Profile | Moisture Content (%) ✖, Fines Content (%) ○, Penetration Resistance (blows/ft) Field N ◆, SPT N_{60} ◇ (0 20 40 60 80 100) | Blows/6" (N bpf) and other Field Data | Sample Type | Sample Number | Lab Tests | Material Description | Water Observations | As-Built

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description	Water Observations
		1 1 3 2 (4) Rec=1.2'	D-1		SILT, soft, dark brown, moist, trace sand, trace organics (roots, charcoal).	
		1 1 2 2 (3) Rec=0.9'	D-2		LEAN CLAY, soft, dark brown with few feo stains, moist, trace gravel, trace sand, trace organics (roots, charcoal, wood).	
5	285	Rec=2.0'	PS-3		LEAN CLAY WITH GRAVEL, subrounded, brown to gray brown, moist.	02-21-21
		3 2 3 (5) Rec=1.1'	D-4		LEAN CLAY, medium stiff, brown to gray brown, moist, homogeneous, trace gravel.	
		1 2 2 (4) Rec=1.2'	D-5		LEAN CLAY, soft, gray, moist, homogeneous, trace fine gravel, trace sand.	
10	280	Rec=1.0'	PS-6		LEAN CLAY, gray, moist.	
		3 4 5 (9) Rec=1.3'	D-7		FAT CLAY, stiff, gray, moist, homogeneous, trace gravel, trace sand.	
		2 3 4 (7) Rec=1.5'	D-8		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel, trace sand.	
15	275	Rec=1.8'	PS-9		FAT CLAY WITH GRAVEL, subrounded, gray, moist.	
		1 2 3 (5) Rec=1.3'	D-10		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel, trace sand.	
		2 2 3 (5) Rec=1.0'	D-11		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel, trace sand.	
20	270	Rec=1.9'	PS-12		FAT CLAY WITH GRAVEL, subrounded, gray, moist.	
		3 5 5 (10) Rec=0.8'	D-13		FAT CLAY, stiff, gray, wet, homogeneous, trace sand.	
		4 5 6 (11) Rec=0.8'	D-14		FAT CLAY, stiff, gray, wet, homogeneous, trace fine gravel.	
25	265	Rec=2.0'	PS-15		FAT CLAY, gray, moist, trace gravel.	

CONTINUED NEXT PAGE *(see last page for notes)*

Project: SR-542/Squalicum Creek to Bellingham Bay - Fish Passage

Job Number: XL6093

Route & MP Range: SR 542 MP 3.43 - 3.47

Depth (feet)	Elevation (feet)	Profile	Moisture Content (%) / Fines Content (%) / Penetration Resistance (blows/ft) Field N, SPT N_{60}	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description	Water Observations	As-Built
				Rec=2.0'		PS-15		FAT CLAY, gray, moist, trace gravel.		
				2 3 5 (8) Rec=1.5'		D-16		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel, trace sand.		
	260			2 3 5 (8) Rec=1.5'		D-17		FAT CLAY, medium stiff, gray, moist, homogeneous, trace gravel, trace sand.		
30				Rec=2.0'		PS-18		FAT CLAY, gray, moist, trace gravel.		

HOLE ENDED AT 31.0 FEET ON 2-22-2021

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter(X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988. Blow counts per 6-inch increment have not been corrected.
4. The groundwater level(s), if shown, represents observations made during drilling and/or stabilized water measured during a bail test. The groundwater level should be considered approximate and will vary based on seasonal and other effects.

BAIL-RECHARGE TEST RESULTS:

Test Date: February 21, 2021
Hole Depth / Casing Depth: 31.0 feet / 29.0 feet
Water Depth Before Bailing: 4.0 feet
Note: will check water in the morning before install

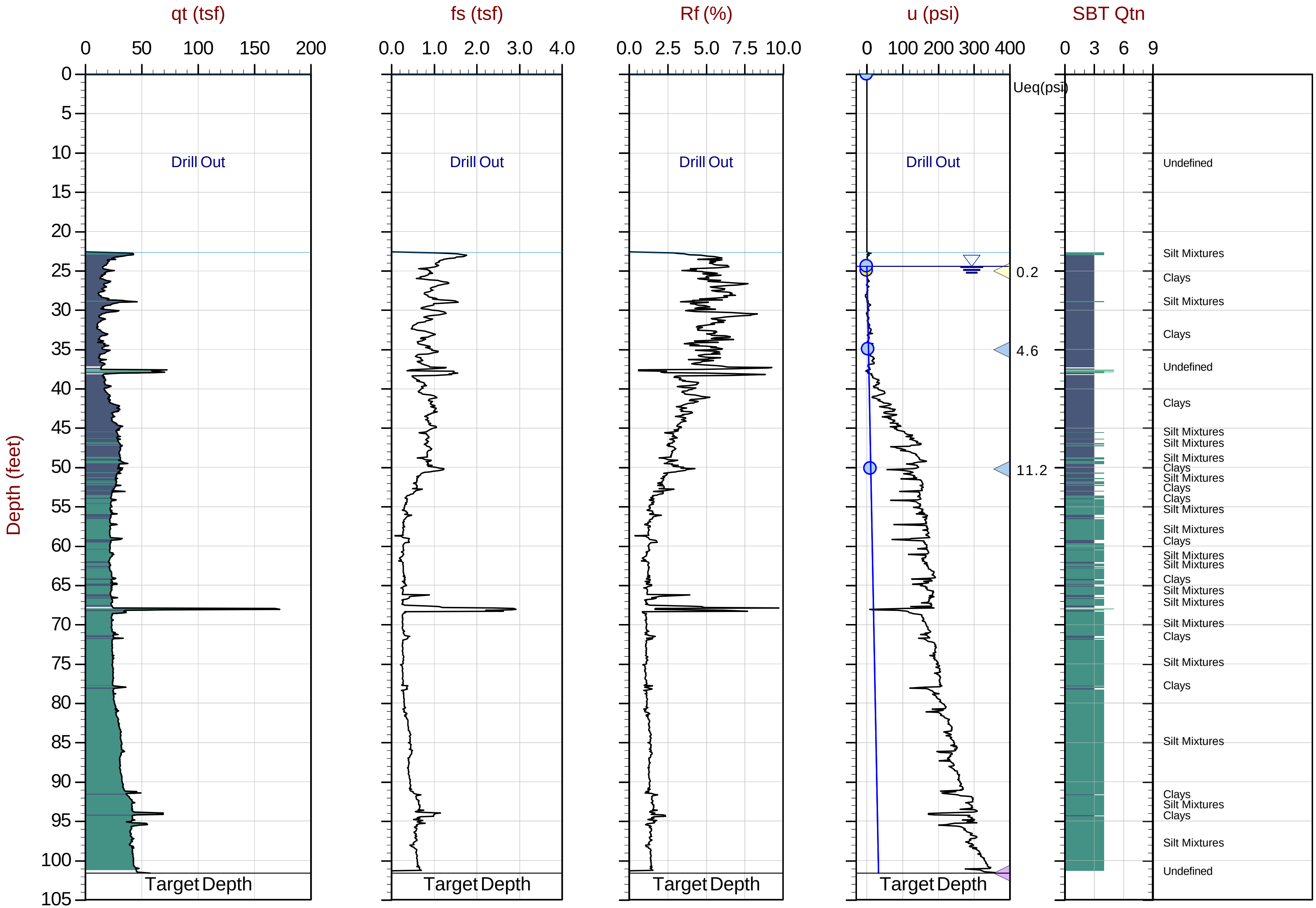
ELAPSED TIME (minutes)	WATER DEPTH (feet)
0	21.5
1	20.6
2	20.4
3	20.4
4	20.3
5	20.3
10	19.1
15	19.1
20	19.1

CONETEC

WSDOT

Job No: 21-59-21784
Date: 2021-03-03 08:25
Site: SR542 MP3.45

Sounding: CPT-1
Cone: EC529



Max Depth: 30.975 m / 101.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-59-21784_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 48.79994 Long: -122.40474

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

APPENDIX B: LABORATORY TEST RESULTS

CONTENTS

LABORATORY TEST RESULTS

The soil samples were classified visually in the field in general accordance with Chapter 4 of the WSDOT GDM. The classification criteria in the GDM is a modified version of ASTM D 2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Once transported to the laboratory, the samples were reexamined, various laboratory tests were performed on selected samples, and the field classifications were modified accordingly. We refined our visual-manual soil classifications based on the results of the laboratory tests, using the Standard Practice for Classification of Soils for Engineering Purposes (ASTM D 2487).

MOISTURE (NATURAL WATER) CONTENT

Natural moisture content determinations were performed in accordance with ASTM D 2216 on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time the explorations are performed and is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of the moisture content determinations are shown on the Laboratory Summary sheets attached at the end of this appendix and are included on the boring logs in Appendix A.

ATTERBERG LIMITS

Atterberg limits were determined on selected samples in accordance with American Association of State Highway and Transportation Officials (AASHTO) T89 and AASHTO T90. This analysis yields index parameters of the soil that are useful in soil classification and analyses, including liquefaction analysis. An Atterberg limits test determines a soil’s liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil’s plasticity index (PI) can be determined by subtracting PL from LL. The test results are shown on the Laboratory Summary sheets attached at the end of this appendix and are included on the boring logs in Appendix A.

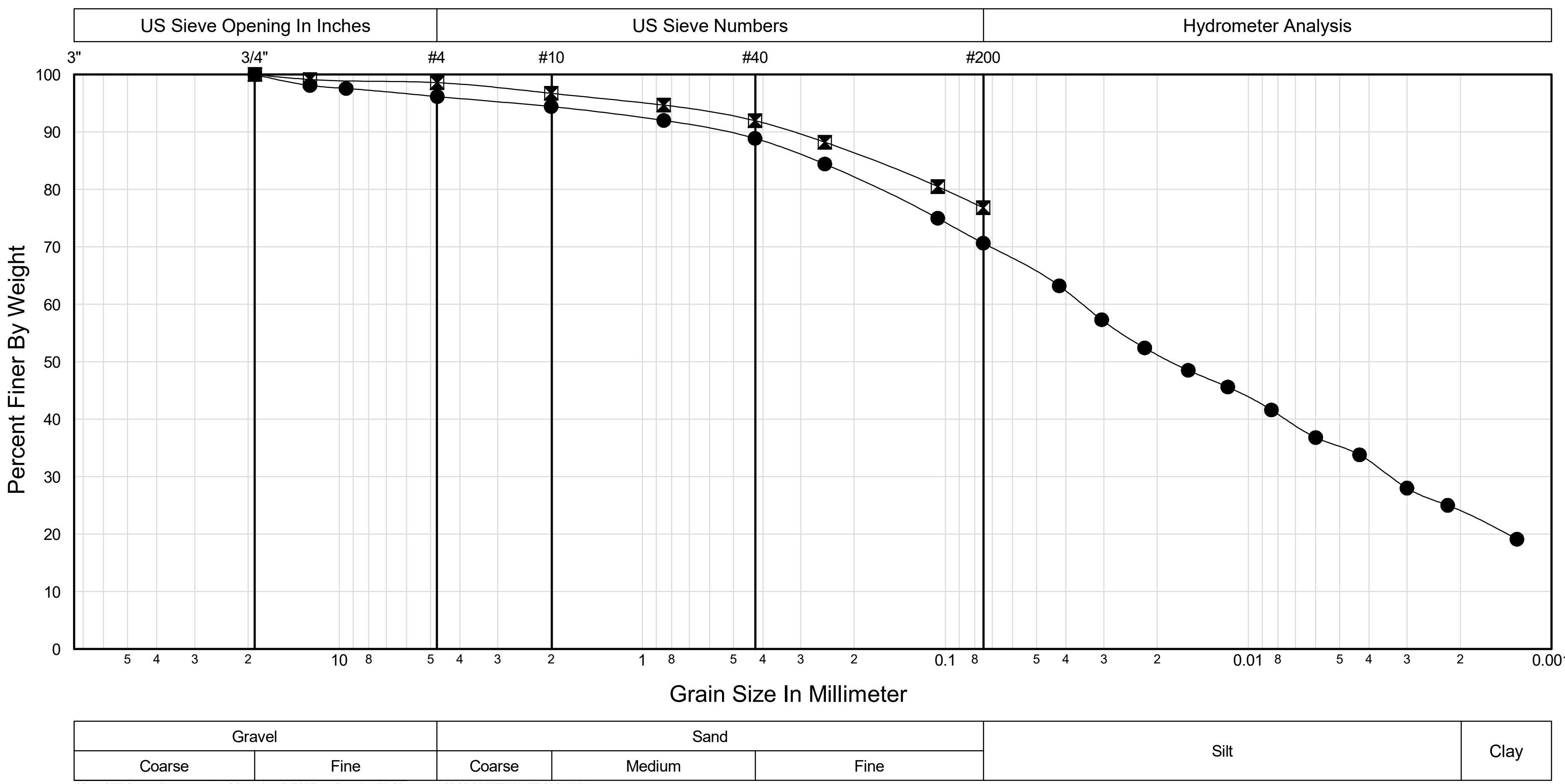
PARTICLE-SIZE ANALYSES

Particle-size analyses were conducted on selected samples to determine their grain-size distributions. Grain-size distributions were determined by sieve analysis in general accordance with AASHTO T27-11. Several grain-size distributions also included a hydrometer analysis by AASHTO T88. The hydrometer analysis yields the grain-size distribution of the sample fraction finer than the U.S. No. 200 mesh sieve. Results of the particle-size analysis are shown on the Laboratory Summary sheets that are attached at the end of this appendix.

GRAIN SIZE ANALYSIS SUMMARY FOR Creek Gorge Surface Sample

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	0.0	GPS Pt 1	CL	LEAN CLAY with SAND	11-30-21	17	30	13	17		2.70	3.9	25.5	70.6			0.035	0.018	0.003	0.001	
⧗	1.0	GPS Pt 2	CL	LEAN CLAY with SAND	11-30-21	8	33	15	18			1.4	21.8	76.8							

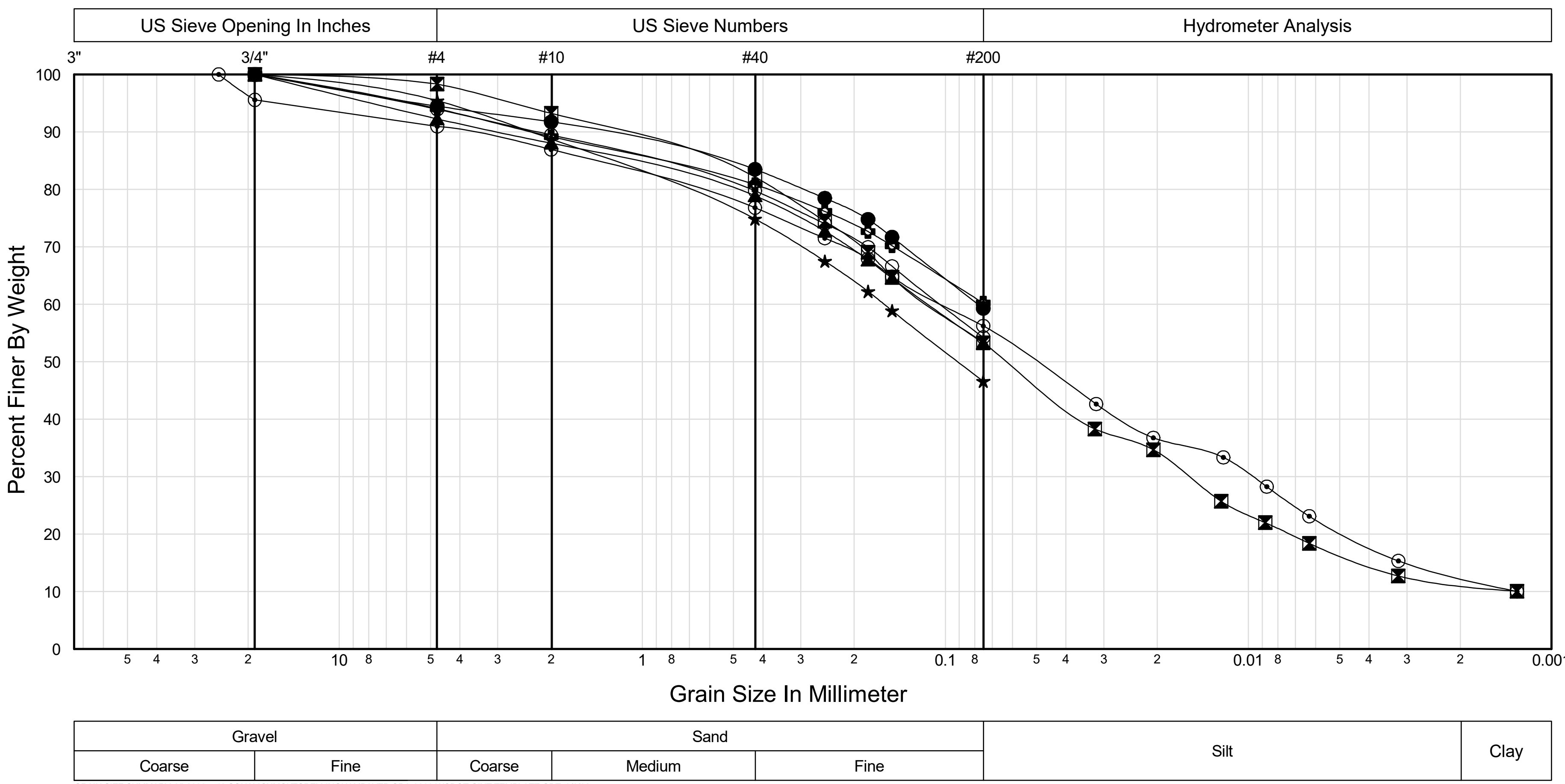


WSDOT

GRAIN SIZE ANALYSIS SUMMARY FOR H-1vw-20

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

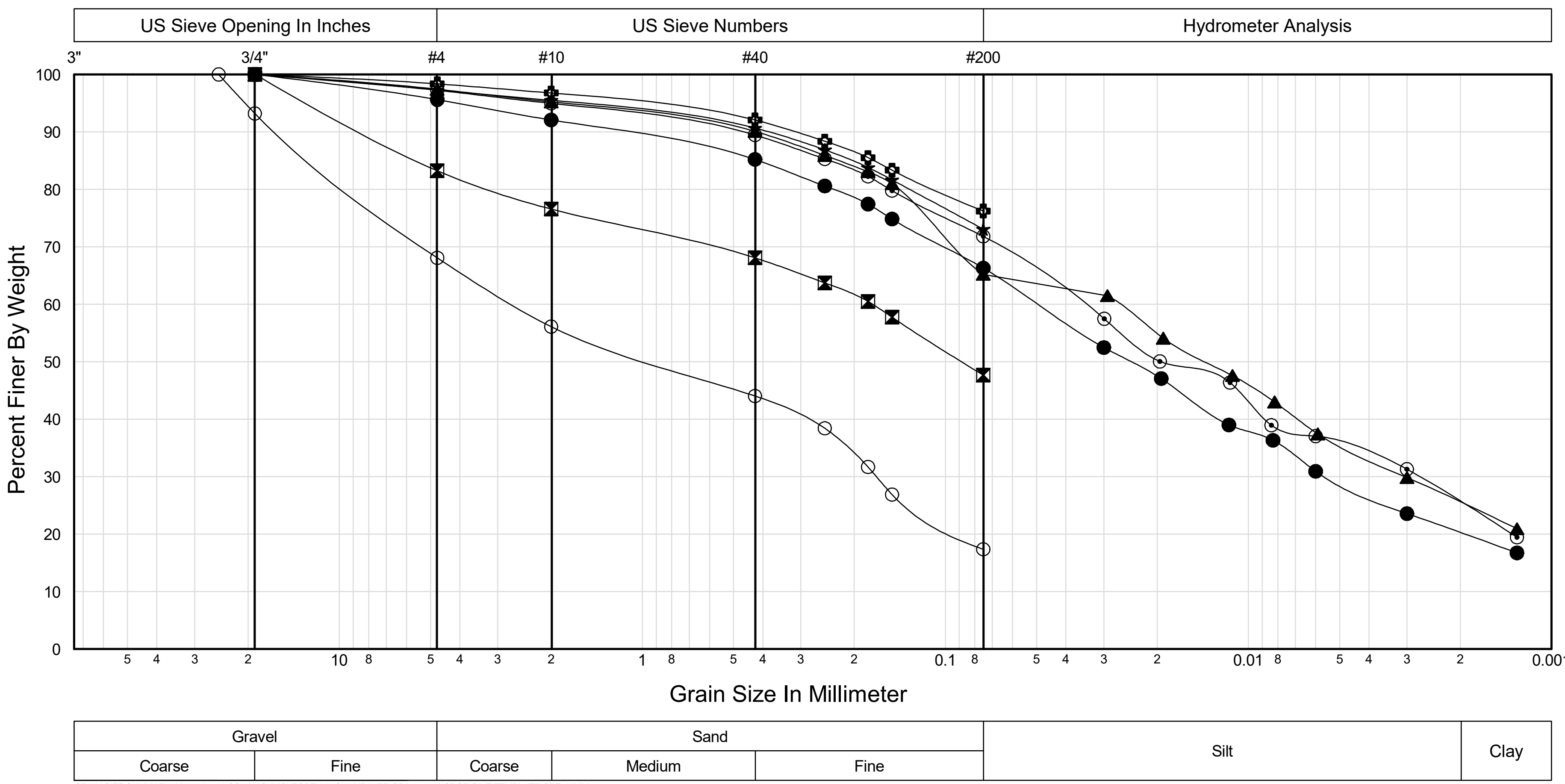
Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	9.0	D-4	CL	SANDY LEAN CLAY	9-21-20	26	32	19	13			5.5	35.2	59.3			0.078				
⊠	10.5	PS-5	CL	SANDY LEAN CLAY	12-4-20	20	26	17	9		2.78	1.7	45.0	53.3			0.113	0.062	0.016	0.007	
▲	12.5	D-6	CL	SANDY LEAN CLAY	9-21-20	21	26	18	8			7.7	39.0	53.2			0.114				
★	17.0	D-8	SM	SILTY SAND	9-21-20	19	n/a	n/a	NP			4.6	48.8	46.6			0.159	0.091			
⊙	19.0	PS-9	CL	SANDY LEAN CLAY	12-24-20	23	27	17	10		2.70	9.0	34.7	56.2			0.102	0.051	0.010	0.005	
✿	21.0	D-10	CL	SANDY LEAN CLAY	9-21-20	19	29	19	10			5.9	33.9	60.2							
○	24.0	D-11	ML	SANDY SILT	9-21-20	20	n/a	n/a	NP			6.1	39.7	54.2			0.104				



GRAIN SIZE ANALYSIS SUMMARY FOR H-1vw-20

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	27.0	D-12	CL	SANDY LEAN CLAY	9-21-20	22	32	18	14		2.79	4.4	29.3	66.3			0.049	0.025	0.006	0.002	
⧗	34.0	D-14	SC	CLAYEY SAND with GRAVEL	9-21-20	27	36	20	16			16.8	35.6	47.7			0.174	0.088			
▲	49.0	D-17	CL	SANDY LEAN CLAY	9-21-20	18	29	15	14		2.77	2.6	32.2	65.2			0.027	0.014	0.003		
★	74.0	D-22	CL	LEAN CLAY with SAND	9-21-20	20	30	17	13			2.7	24.2	73.1							
⊙	75.5	PS-23	CL	LEAN CLAY with SAND	12-24-20	19	35	18	17		2.77	2.8	25.4	71.8			0.035	0.019	0.003	0.001	
✥	81.0	D-25	CL	LEAN CLAY with SAND	9-21-20	20	33	16	17			1.6	22.1	76.2							
○	104.0	D-29	SM	SILTY SAND with GRAVEL	9-21-20	9	n/a	n/a	NP			31.9	50.7	17.4			2.650	0.914	0.169	0.091	

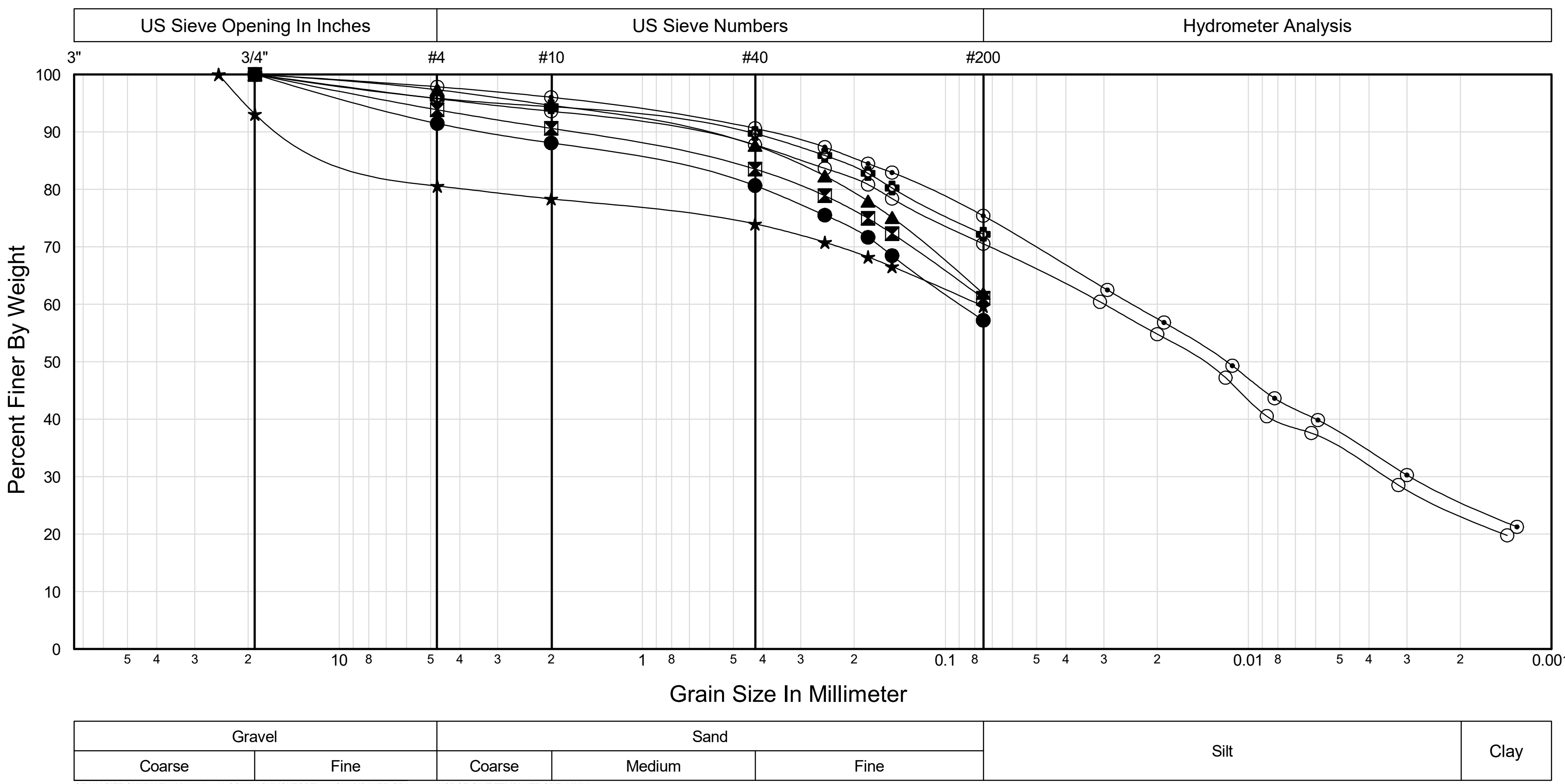


WSDOT

GRAIN SIZE ANALYSIS SUMMARY FOR H-2p-20

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	14.0	D-4	CL	SANDY LEAN CLAY	9-21-20	27	31	18	13			8.5	34.3	57.2			0.089				
⧗	37.0	D-10	CL	SANDY LEAN CLAY	9-21-20	23	31	19	12			6.2	32.8	61.0							
▲	47.0	D-14	CL	SANDY LEAN CLAY	9-21-20	23	31	19	12			2.7	35.5	61.9							
★	54.0	D-17	CL	SANDY LEAN CLAY with GRAVEL	9-21-20	21	32	16	16			19.4	20.9	59.7			0.078				
⊙	59.0	D-19	CL	LEAN CLAY with SAND	9-21-20	8	30	16	14		2.76	2.2	22.4	75.4			0.024	0.012	0.003		
✪	69.0	D-23	CL	LEAN CLAY with SAND	9-21-20	21	30	17	13			4.2	23.7	72.2							
○	74.0	PS-25	CL	LEAN CLAY with SAND	11-10-20	21	34	16	18		2.60	4.2	25.3	70.5			0.030	0.014	0.004	0.001	



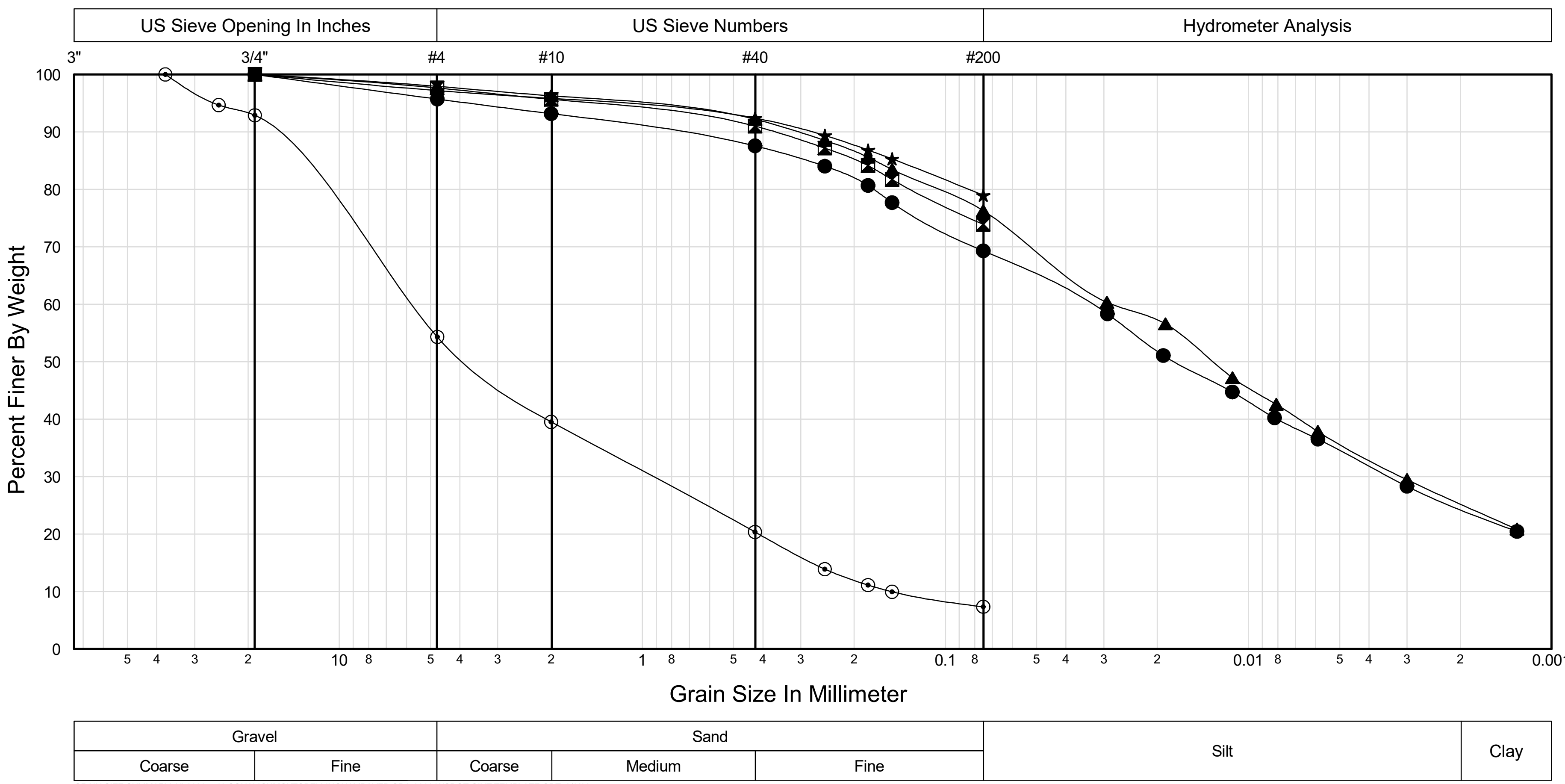
GRAIN SIZE SUMMARY XL6093_542 SQUALICUMCKTOBELLINGHAMBAYFP.GPJ 2020 WSDOT GINT TEMPLATE.GDT 3/8/22

WSDOT

GRAIN SIZE ANALYSIS SUMMARY FOR H-2p-20

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

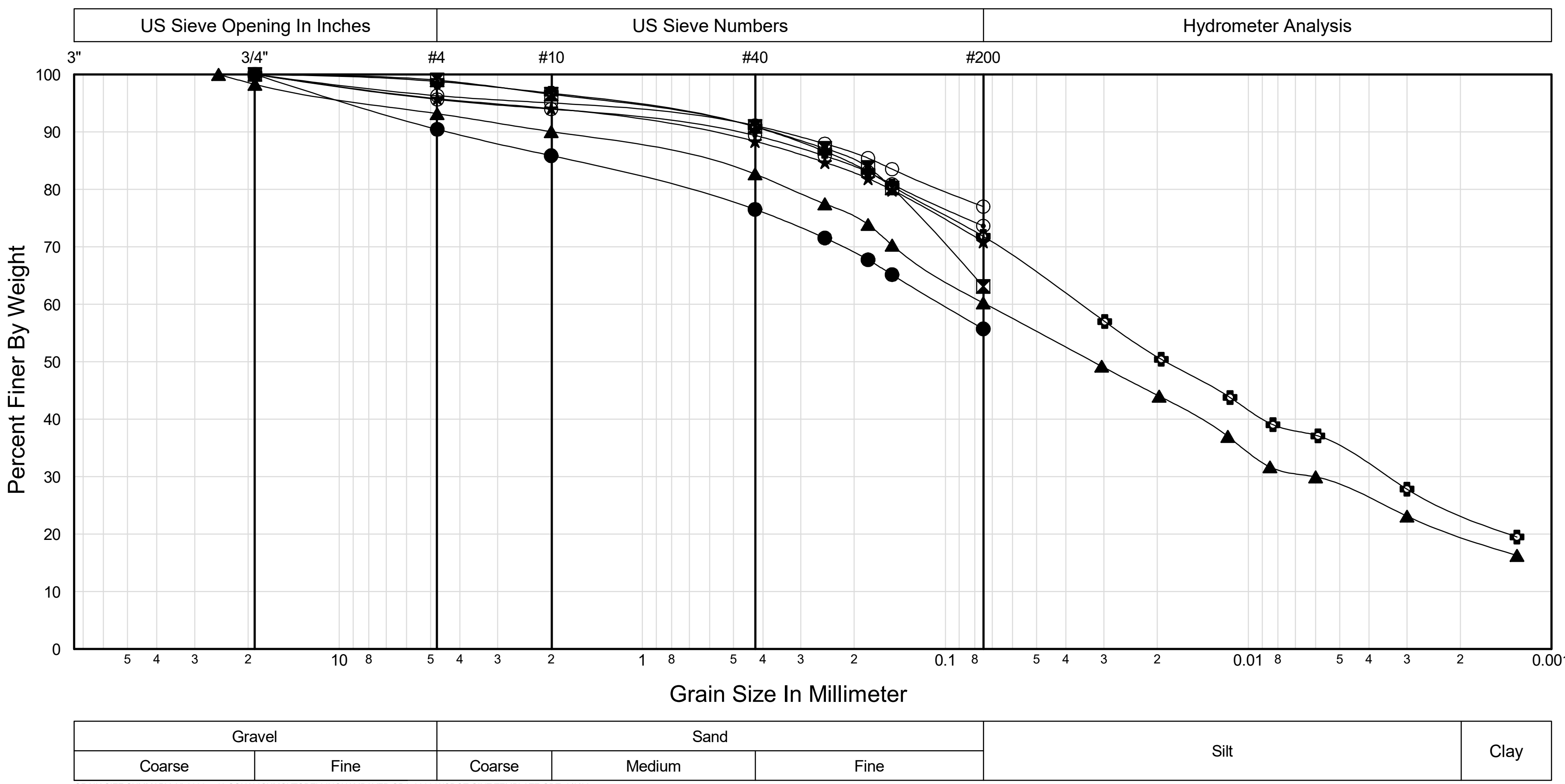
Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	76.0	D-26	CL	SANDY LEAN CLAY	9-21-20	20	30	16	14		2.80	4.3	26.4	69.3			0.034	0.017	0.003		
⧗	84.0	D-28	CL	LEAN CLAY with SAND	9-21-20	19	32	15	17			2.4	23.8	73.9							
▲	86.0	PS-29	CL	LEAN CLAY with SAND	11-16-20	21	38	17	21		2.80	2.1	21.7	76.3			0.028	0.013	0.003		
★	89.0	D-30	CL	LEAN CLAY with SAND	9-21-20	22	36	18	18			2.8	18.2	79.0							
⊙	104.0	D-33	SP-SM	POORLY GRADED SAND with SILT and GRAVEL	9-21-20	6	n/a	n/a	NP			45.7	47.0	7.3	1.0	39	5.826	3.690	0.926	0.412	0.151



GRAIN SIZE ANALYSIS SUMMARY FOR H-3vw-20

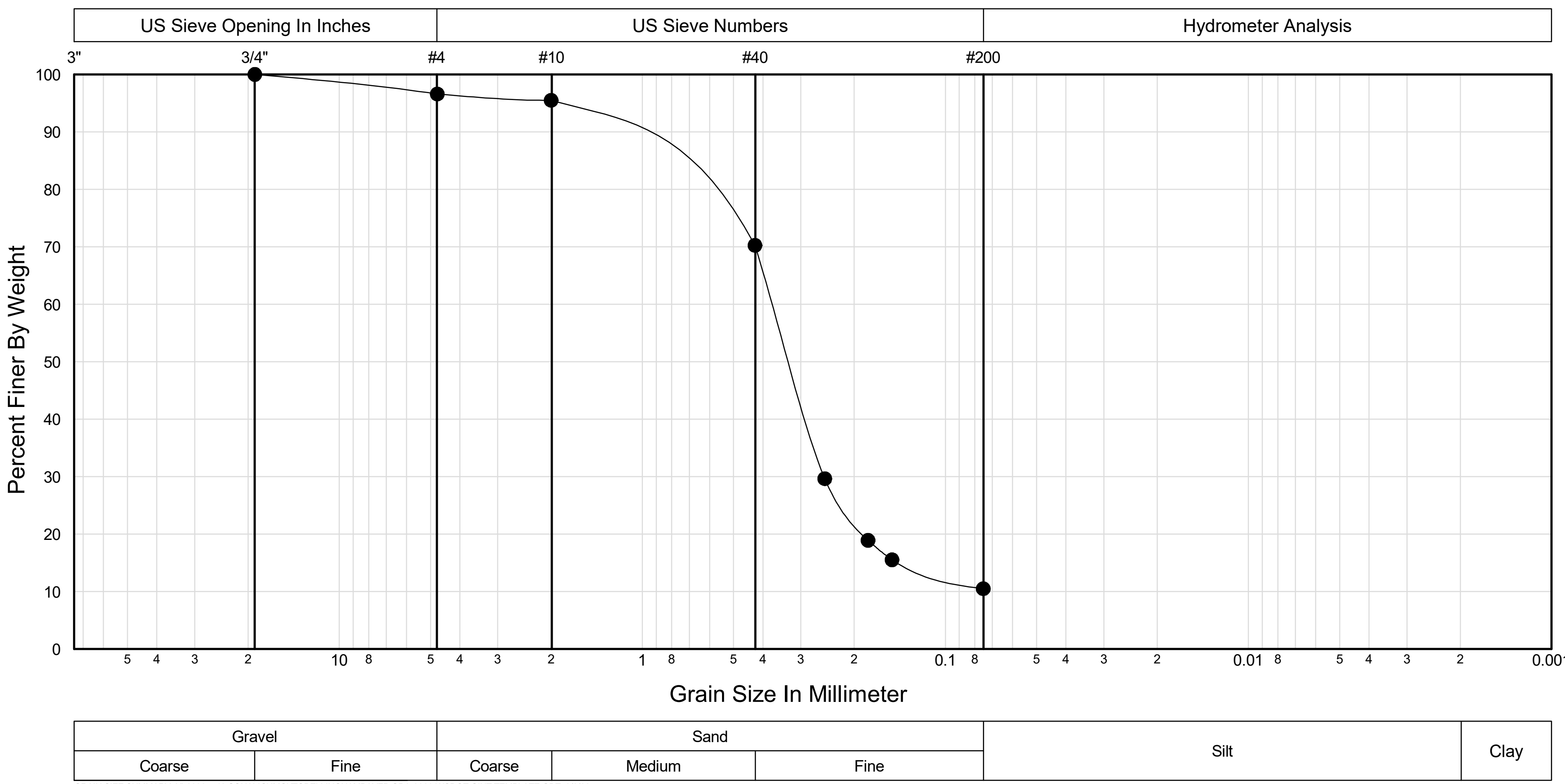
Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	17.5	D-7	ML	SANDY SILT	9-21-20	21	21	n/a	NP			9.6	34.7	55.7			0.103				
⊠	22.5	D-9	CL	SANDY LEAN CLAY	9-21-20	27	39	22	17			1.0	35.9	63.1							
▲	32.5	D-13	CL	SANDY LEAN CLAY	9-21-20	18	31	19	12		2.78	6.8	32.9	60.2			0.074	0.033	0.006	0.002	
★	39.5	D-16	CL	LEAN CLAY with SAND	9-21-20	18	32	17	15			4.2	24.9	70.9							
⊙	54.5	D-19	CL	LEAN CLAY with SAND	9-21-20	20	30	16	14			4.3	22.0	73.6							
✿	69.5	D-22	CL	LEAN CLAY with SAND	9-21-20	20	30	15	15		2.79	1.2	27.0	71.8			0.036	0.019	0.004	0.001	
○	89.5	D-26	CL	LEAN CLAY with SAND	9-21-20	21	35	17	18			3.7	19.3	77.0							



Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**

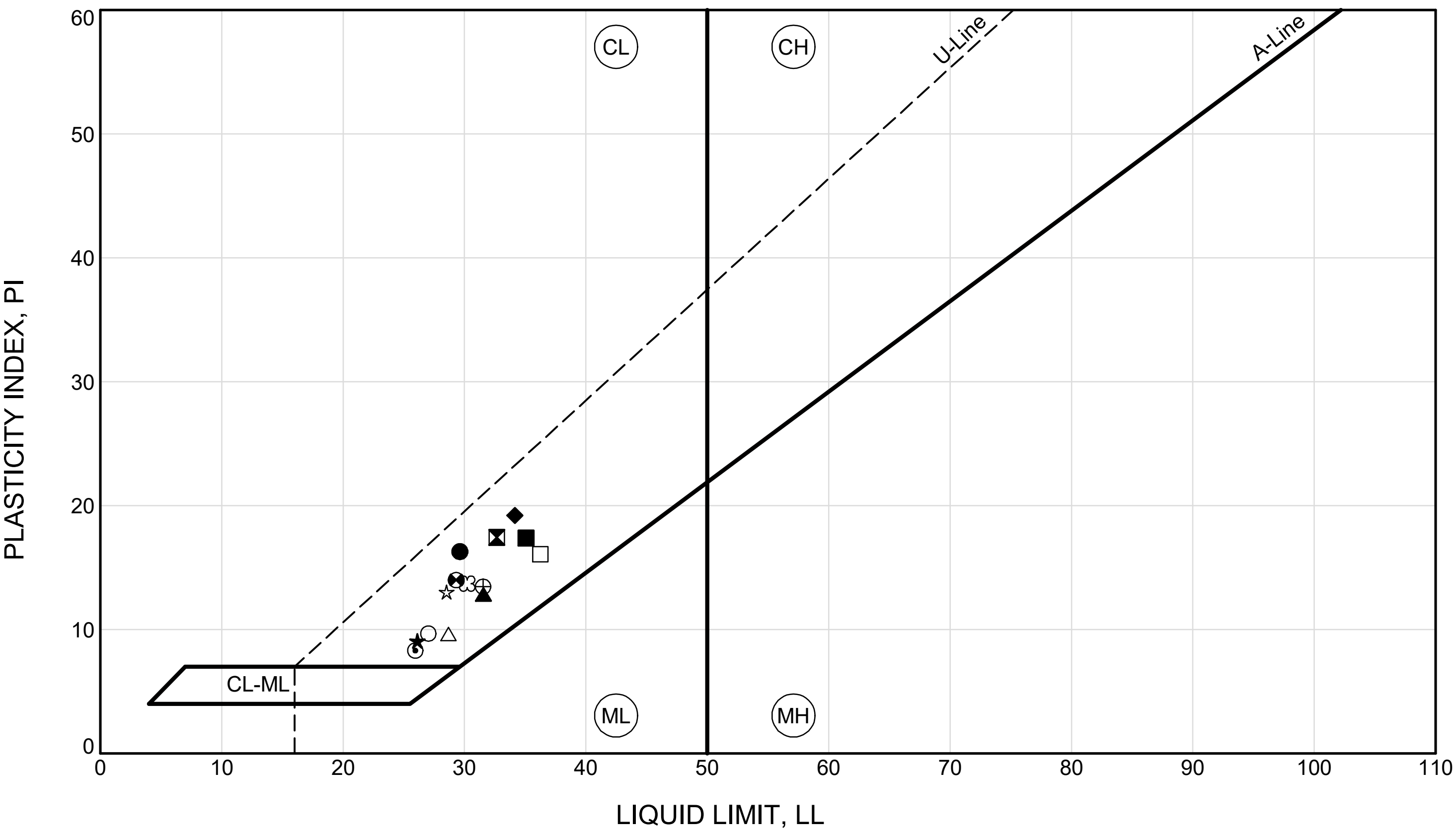
Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	114.5	D-31	SP-SM	POORLY GRADED SAND with SILT	9-21-20	20	n/a	n/a	NP			3.4	86.1	10.5	2.4	5	0.372	0.326	0.251	0.186	



WSDOT

ATTERBERG LIMITS SUMMARY

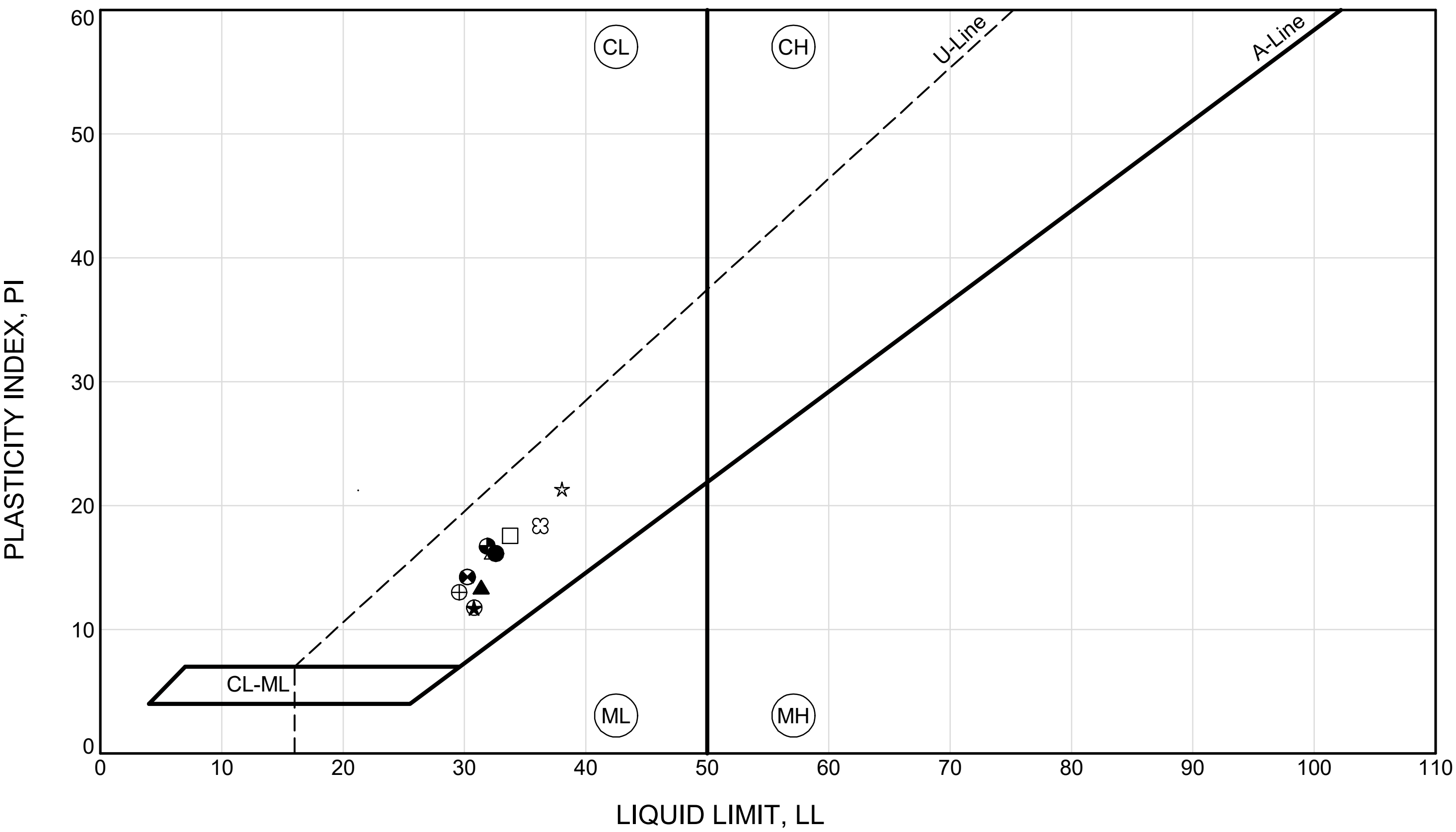
Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**



Symbol	Hole No.	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Fines (%)	Silt (%)	Clay (%)
●	Creek Gorge Surface Sample	0.0	GPS Pt 1	CL	LEAN CLAY with SAND	11-30-21	17	30	13	17	70.6	46.9	23.7
⊠	Creek Gorge Surface Sample	1.0	GPS Pt 2	CL	LEAN CLAY with SAND	11-30-21	8	33	15	18	76.8		
▲	H-1vw-20	9.0	D-4	CL	SANDY LEAN CLAY	9-21-20	26	32	19	13	59.3		
★	H-1vw-20	10.5	PS-5	CL	SANDY LEAN CLAY	12-4-20	20	26	17	9	53.3	42.2	11.0
⊙	H-1vw-20	12.5	D-6	CL	SANDY LEAN CLAY	9-21-20	21	26	18	8	53.2		
	H-1vw-20	17.0	D-8	SM	SILTY SAND		19	n/a	n/a	NP	46.6		
○	H-1vw-20	19.0	PS-9	CL	SANDY LEAN CLAY	12-24-20	23	27	17	10	56.2	44.3	12.0
△	H-1vw-20	21.0	D-10	CL	SANDY LEAN CLAY	9-21-20	19	29	19	10	60.2		
	H-1vw-20	24.0	D-11	ML	SANDY SILT		20	n/a	n/a	NP	54.2		
⊕	H-1vw-20	27.0	D-12	CL	SANDY LEAN CLAY	9-21-20	22	32	18	14	66.3	46.8	19.5
□	H-1vw-20	34.0	D-14	SC	CLAYEY SAND with GRAVEL	9-21-20	27	36	20	16	47.7		
⊗	H-1vw-20	49.0	D-17	CL	SANDY LEAN CLAY	9-21-20	18	29	15	14	65.2	40.6	24.6
	H-1vw-20	59.0	D-19		MC Only		18	n/a	n/a	NP			
☆	H-1vw-20	64.0	D-20		MC & AL Only	9-21-20	20	29	15	14			
✿	H-1vw-20	74.0	D-22	CL	LEAN CLAY with SAND	9-21-20	20	30	17	13	73.1		
■	H-1vw-20	75.5	PS-23	CL	LEAN CLAY with SAND	12-24-20	19	35	18	17	71.8	47.5	24.3
◆	H-1vw-20	79.0	PS-24		LEAN CLAY	7-7-21		34	15	19			

ABBREVIATIONS:
LL = liquid limit; MC = moisture content; n/a = test attempted; NP = nonplastic; PI = plasticity index; PL = plastic limit; USCS = Unified Soil Classification System code
USCS codes listed on graph: CL = lean clay; CH = fat clay; ML = silt; MH = elastic silt; CL-ML = silty clay

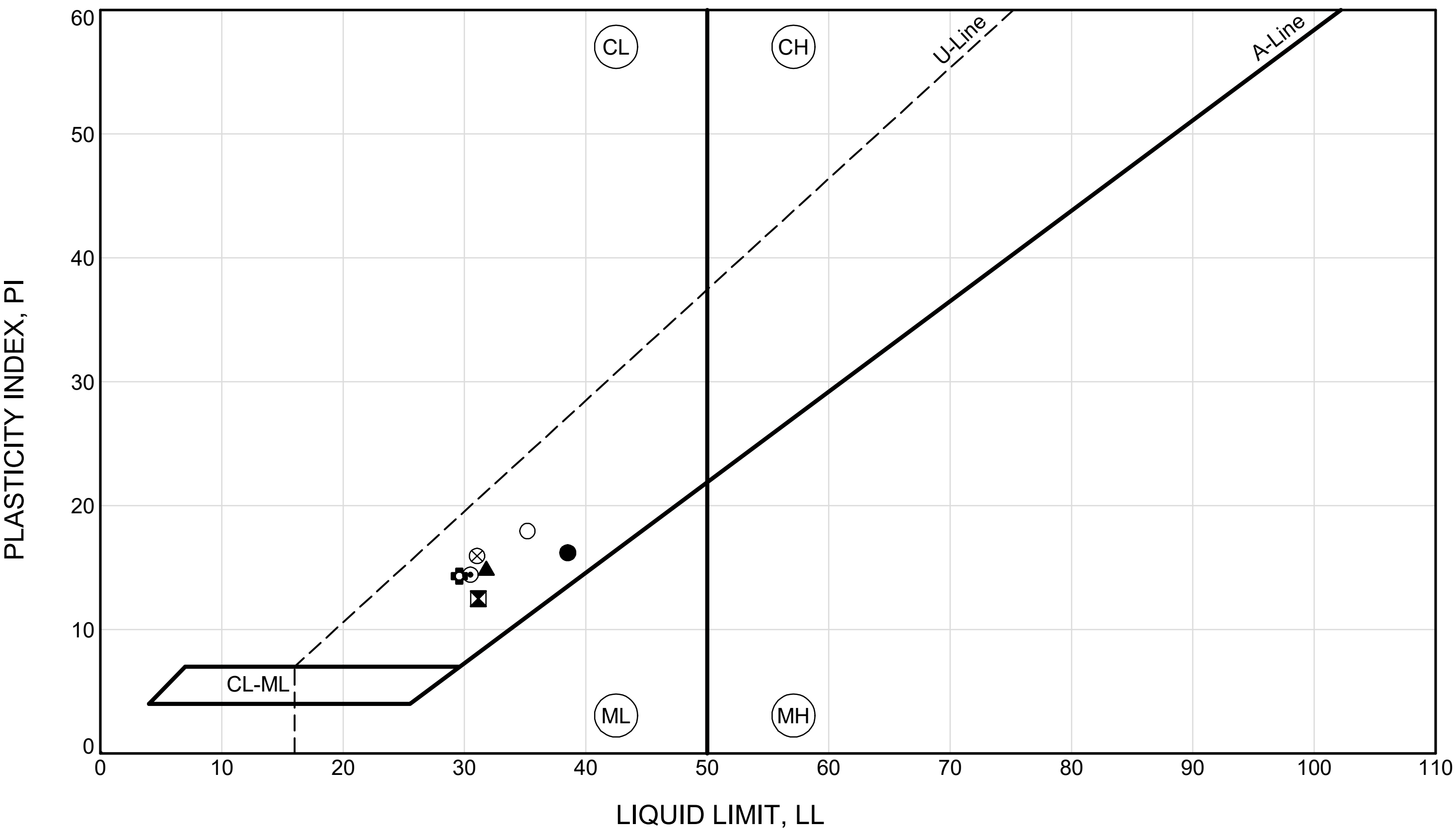
Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**



Symbol	Hole No.	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Fines (%)	Silt (%)	Clay (%)
●	H-1vw-20	81.0	D-25	CL	LEAN CLAY with SAND	9-21-20	20	33	16	17	76.2		
	H-1vw-20	104.0	D-29	SM	SILTY SAND with GRAVEL		9	n/a	n/a	NP	17.4		
▲	H-2p-20	14.0	D-4	CL	SANDY LEAN CLAY	9-21-20	27	31	18	13	57.2		
★	H-2p-20	37.0	D-10	CL	SANDY LEAN CLAY	9-21-20	23	31	19	12	61.0		
⊙	H-2p-20	47.0	D-14	CL	SANDY LEAN CLAY	9-21-20	23	31	19	12	61.9		
	H-2p-20	49.0	D-15		MC Only		23	n/a	n/a	NP			
	H-2p-20	52.0	D-16		MC Only		16	n/a	n/a	NP			
△	H-2p-20	54.0	D-17	CL	SANDY LEAN CLAY with GRAVEL	9-21-20	21	32	16	16	59.7		
⊗	H-2p-20	59.0	D-19	CL	LEAN CLAY with SAND	9-21-20	8	30	16	14	75.4	50.4	25.0
⊕	H-2p-20	69.0	D-23	CL	LEAN CLAY with SAND	9-21-20	21	30	17	13	72.2		
□	H-2p-20	74.0	PS-25	CL	LEAN CLAY with SAND	11-10-20	21	34	16	18	70.5	47.8	22.7
◍	H-2p-20	76.0	D-26	CL	SANDY LEAN CLAY	9-21-20	20	30	16	14	69.3	45.6	23.7
◕	H-2p-20	84.0	D-28	CL	LEAN CLAY with SAND	9-21-20	19	32	15	17	73.9		
☆	H-2p-20	86.0	PS-29	CL	LEAN CLAY with SAND	11-16-20	21	38	17	21	76.3	51.9	24.4
⌘	H-2p-20	89.0	D-30	CL	LEAN CLAY with SAND	9-21-20	22	36	18	18	79.0		
	H-2p-20	104.0	D-33	SP-SM	POORLY GRADED SAND with SILT and GRAVEL		6	n/a	n/a	NP	7.3		
	H-3vw-20	17.5	D-7	ML	SANDY SILT	9-21-20	21	21	n/a	NP	55.7		

ABBREVIATIONS:
LL = liquid limit; MC = moisture content; n/a = test attempted; NP = nonplastic; PI = plasticity index; PL = plastic limit; USCS = Unified Soil Classification System code
USCS codes listed on graph: CL = lean clay; CH = fat clay; ML = silt; MH = elastic silt; CL-ML = silty clay

Job No: **XL6093**
Project: **SR-542/Squalicum Creek to Bellingham Bay - Fish Passage**



Symbol	Hole No.	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Fines (%)	Silt (%)	Clay (%)
●	H-3vw-20	22.5	D-9	CL	SANDY LEAN CLAY	9-21-20	27	39	22	17	63.1		
⧗	H-3vw-20	32.5	D-13	CL	SANDY LEAN CLAY	9-21-20	18	31	19	12	60.2	41.1	19.1
▲	H-3vw-20	39.5	D-16	CL	LEAN CLAY with SAND	9-21-20	18	32	17	15	70.9		
	H-3vw-20	49.5	D-18		MC Only		18	n/a	n/a	NP			
⊙	H-3vw-20	54.5	D-19	CL	LEAN CLAY with SAND	9-21-20	20	30	16	14	73.6		
✿	H-3vw-20	69.5	D-22	CL	LEAN CLAY with SAND	9-21-20	20	30	15	15	71.8	48.9	22.9
○	H-3vw-20	89.5	D-26	CL	LEAN CLAY with SAND	9-21-20	21	35	17	18	77.0		
	H-3vw-20	114.5	D-31	SP-SM	POORLY GRADED SAND with SILT	9-21-20	20	n/a	n/a	NP	10.5		
⊗	H-4si-21	9.0	PS-6		LEAN CLAY	7-7-21		31	15	16			

ABBREVIATIONS:
LL = liquid limit; MC = moisture content; n/a = test attempted; NP = nonplastic; PI = plasticity index; PL = plastic limit; USCS = Unified Soil Classification System code
USCS codes listed on graph: CL = lean clay; CH = fat clay; ML = silt; MH = elastic silt; CL-ML = silty clay

APPENDIX C: ADVANCED TESTING

CONTENTS

ADVANCED TESTING

Advanced laboratory testing was performed on select undisturbed samples. The following sections describe the laboratory testing procedures used for this project.

CRS CONSOLIDATION TESTING

The one-dimensional CRS consolidation test provides data for estimating settlement. The CRS testing was done in general accordance with ASTM D 4186. After the sample was extracted, a relatively undisturbed, fine-grained sample was carefully trimmed and fit into a rigid steel ring with porous stones placed on the top and bottom of the sample to allow drainage. Vertical strain was then applied continuously to the sample in a way that allowed the sample to partially consolidate under the given strain rate. Measurements were made of the compression of the sample over time, the total load placed on the sample, and the excess pore pressure at the base of the sample throughout the test. Rebound was measured during the unloading phase. In general, an excess pore pressure ratio of 3 percent is targeted during loading, with an allowance of up to 15 percent without significant concerns about strain rate effects. The test results are plotted in terms of axial strain and coefficient of consolidation versus applied load (stress). The specific gravity of the samples was determined by ASTM D 854. The test results are attached at the end of this appendix.

Sample quality was established using methods by both Lunne et al. (2006) and Terzaghi et al. (1996) as described in Exhibit C-1 and C-2. As needed for sample quality designation (SQD), the preconsolidation stress was estimated using the Casagrande and strain energy methods. *In situ* vertical effective stress was estimated based on the provided soil profile information.

EXHIBIT C-1: SAMPLE QUALITY BY LUNNE ET AL. (2006)

OCR	$\Delta e/e_o$ at σ'_{vo}			
1 to 2	< 0.04	0.04 – 0.07	0.07 – 0.14	> 0.14
2 to 4	< 0.03	0.03 – 0.05	0.05 – 0.10	> 0.10
Quality *	1	2	3	4

NOTE:
* 1 = very good to excellent, 2 = fair to good, 3 = poor, 4 = very poor

EXHIBIT C-2: SAMPLE QUALITY BY TERZAGHI ET AL. (2006)

ε_v at σ'_{vo}	< 1	1 – 2	2 – 4	4 – 8	< 8
Quality *	A	B	C	D	E

NOTE:
* A (best) to E (worst)

Though we make every effort to refine our estimates, the user of this data should apply their own interpretation and engineering judgement to the consolidation test results. Our interpretations are intended solely for the purpose of SQD. SQD approaches are most applicable to low to medium plasticity clays with overconsolidation ratios (OCR) ranging between about 1 and 4. The sample information and results for each test are shown on Figures C-1 to C-4.

DSS Testing

The DSS test estimates the static strength of the soil and was done in general accordance with ASTM D 6527. A relatively undisturbed fine-grained sample was trimmed to a length of about 6 inches, encased in a rubber membrane, and placed in the triaxial cell. With the sample in the test cell, an all-around pressure was applied hydraulically. The sample was allowed to consolidate under the applied pressure with drainage occurring through porous stones through slotted filter paper placed around the sample. When consolidation was completed, the sample was sheared at a constant strain rate under constant volume conditions. The input parameters for DSS tests are summarized in Exhibit C-3.

Exhibit C-3: Input Parameters for DSS Tests

Boring	Sample No.	Figure Number	σ'_{v_max} (psi)[1]	σ'_{vc} (psi)[2]	Shear Rate (inches/hr)
H-2p-20	PS-29	C-5	150	75	0.01
H-4si-21	PS-6	C-6	208	19.6	0.01
H-4si-21	PS-18	C-7	139	38	0.01

NOTES:

1. Highest vertical effective stress in consolidation phase.
2. Effective vertical stress at the end of consolidation phase.

The sample information and results for each test are shown on Figures C-5 to C-7.

Stress-Controlled CDSS Testing

The stress-controlled CDSS test with pore pressure measured provides data for evaluating the liquefaction susceptibility of soils and post cyclic soil strength. The CDSS testing was done in general accordance with ASTM D 8296-19. After the sample was extracted from the Shelby tube, a relatively undisturbed fine-grained sample was carefully trimmed to a height of about 1 inch and a diameter of 2.5 inches, encased in a rubber membrane, and placed in the test cell. With the sample in the test cell, an all-around pressure was applied hydraulically. The sample was allowed to consolidate under the applied pressure with drainage occurring through porous stones through slotted filter paper placed around the sample. After the consolidation process completed, cyclic horizontal shear stresses (τ_{cyc}) were applied sinusoidally at an amplitude of the prescribed stress ratio. The τ_{cyc} was applied under constant volume conditions in a specified frequency of 0.2 Hz (i.e., 5 seconds period per cycle) for a maximum of 30 cycles (corresponding to the anticipated design earthquake magnitude), or the maximum peak-peak strain of 20 percent or the maximum pressure ratio of 0.8 is reached, whichever occurs first. The input parameters for CDSS tests are summarized in Exhibit C-4.

EXHIBIT C-4: INPUT PARAMETERS FOR CDSS TESTS

Boring	Sample No.	Figure Number	σ'_{v_max} (psi)[1]	σ'_{vc} (psi)[2]	Cyclic Stress Ratio
H-2p-20	PS-29	C-8	150	75	0.31
H-4si-21	PS-6	C-9	208	19.6	0.45

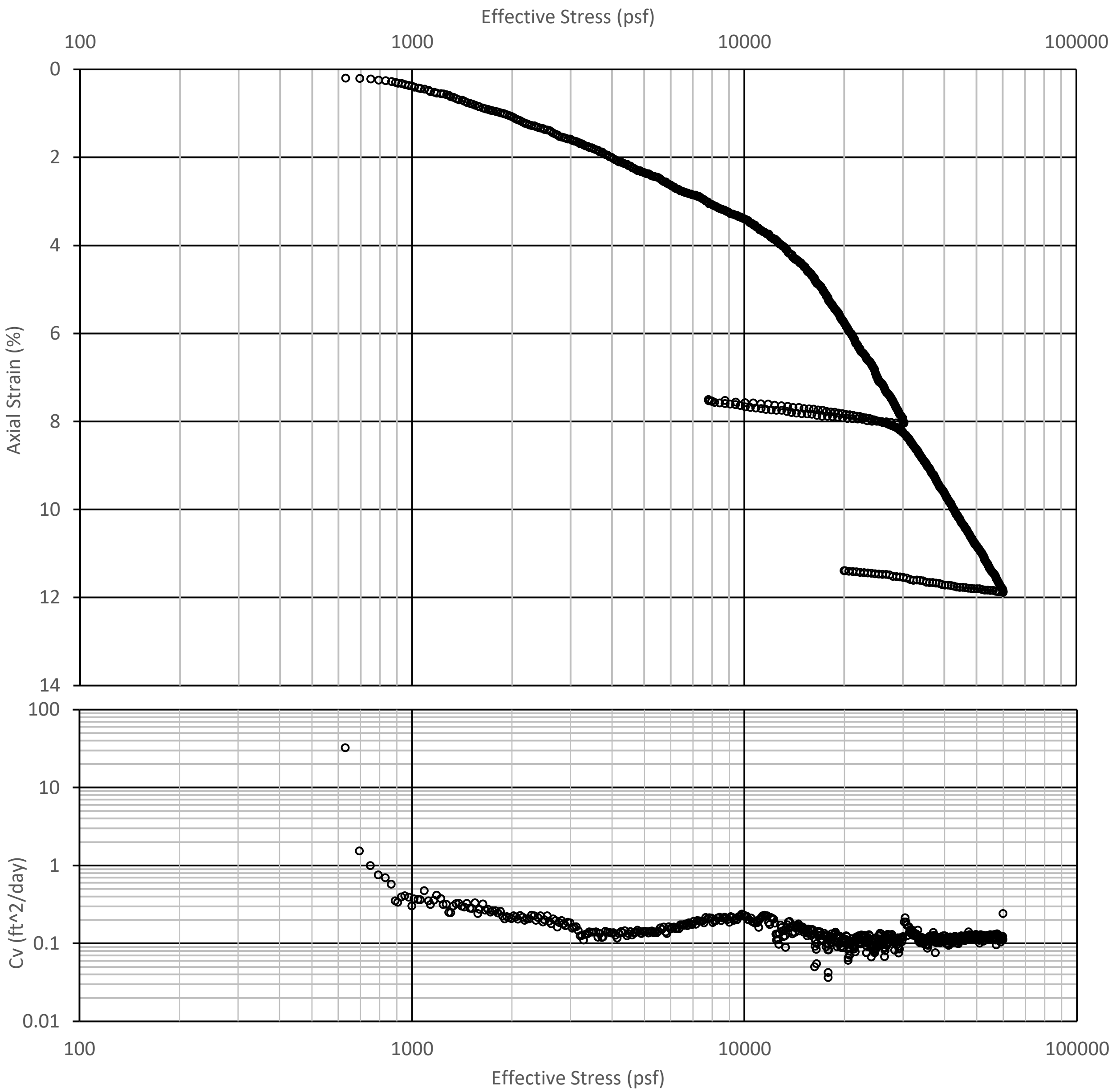
NOTES:

1. Highest vertical effective stress in consolidation phase.
2. Effective vertical stress at the end of consolidation phase.

Once the cyclic loading process is completed without allowing any dissipation of the final excess pore pressure, the sample was then sheared at a constant strain rate in accordance with the general procedure described in ASTM D6528. The sample information and results for each test are shown on Figures C-8 to C-9.

TRIAXIAL TESTING

Triaxial compression tests were performed on several intact fine-grained samples. The consolidated undrained (CU) tests were performed in general accordance with ASTM D 4767. The CU tests with pore pressure measurements included specimens IsoCUs. The sample information and results for each test are shown on Figures C-10 to C-11.



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
76.7	18	14	35	18	17	Lean Clay with Sand	CL

σ'v$_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
7771	16000	15000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.01
Total Unit Weight (pcf)	132.23
Degree of Saturation (%)	98.90
Void Ratio (e0)	0.470

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for H-1vw-20 PS-23 CRS Consolidation

Job Number: 19501-27 5/25/2021

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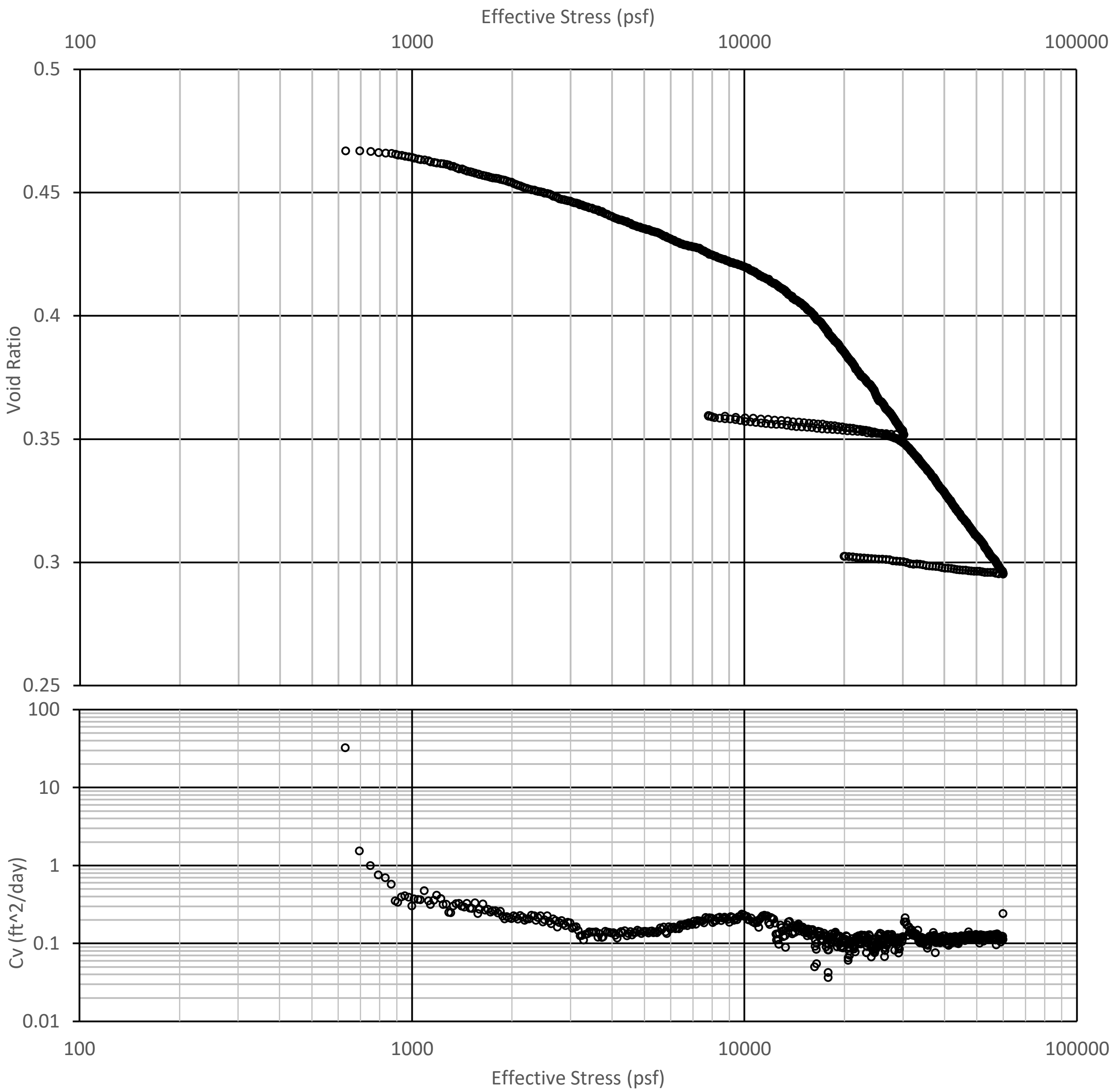
Figure
C-1-1

ML 7/18/2022 \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\Deliverables\In-Basket\FINAL Report\Attachments\Appendices\Appendix B & C - Laboratory Data\Appendix C\19501-27 H-1wp-20 PS-23 CRS.xlsm



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
76.7	18	14	35	18	17	Lean Clay with Sand	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
7771	16000	15000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.01
Total Unit Weight (pcf)	132.23
Degree of Saturation (%)	98.90
Void Ratio (e0)	0.470

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for H-1vw-20 PS-23 CRS Consolidation

Job Number: 19501-27 5/25/2021

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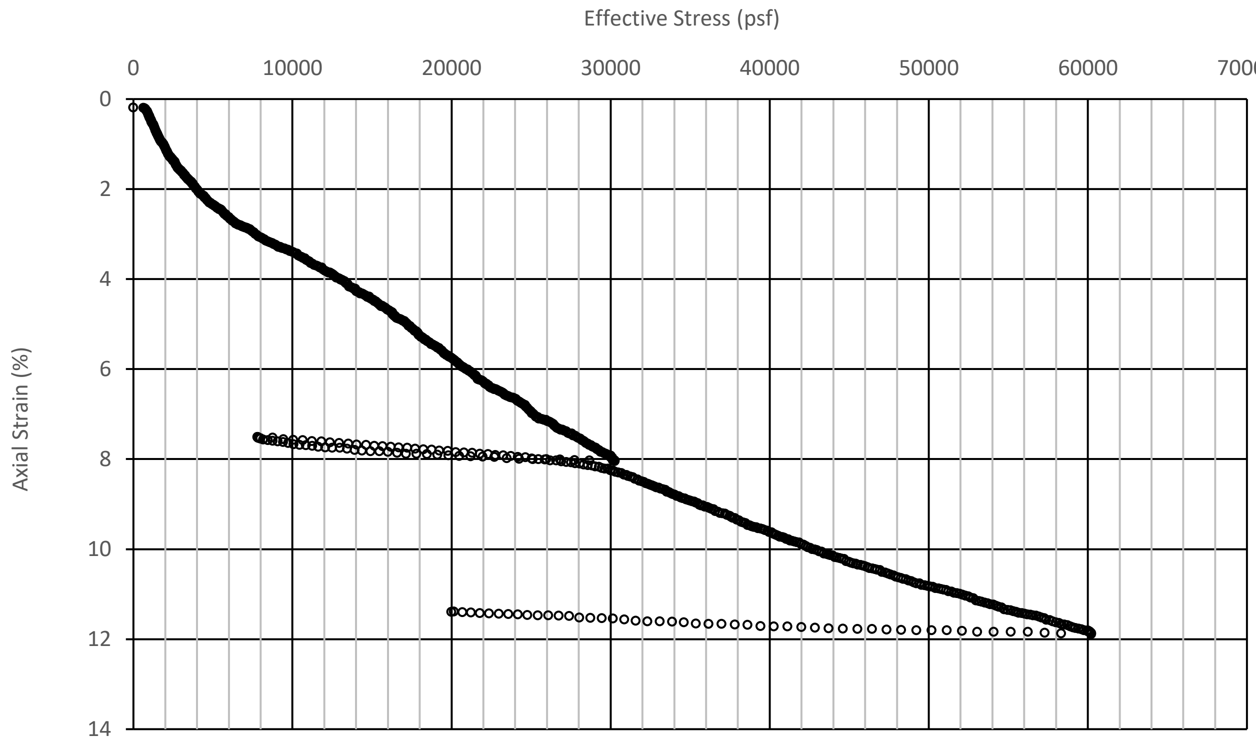
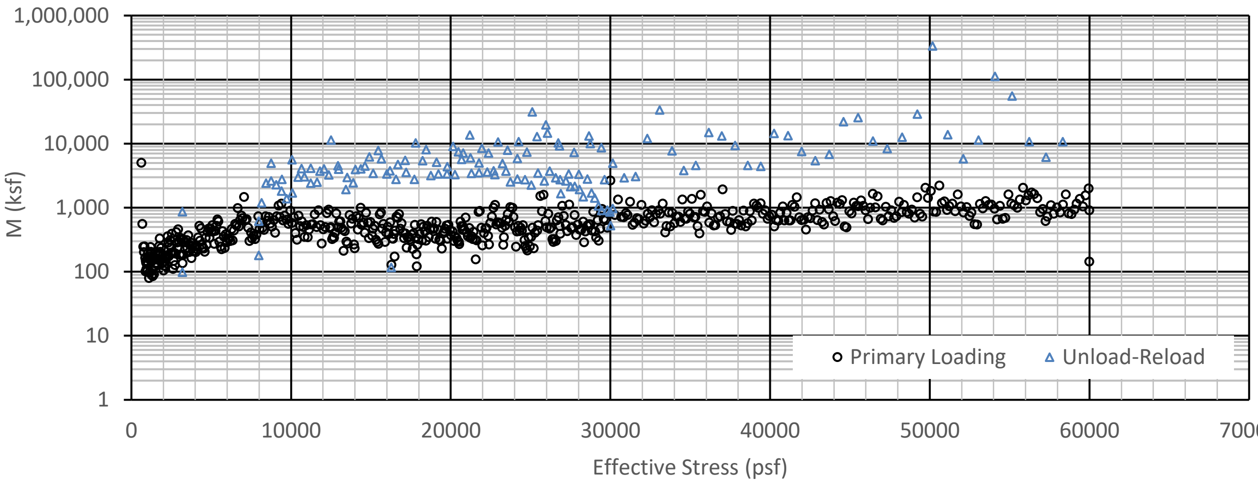
Figure
C-1-2





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
76.7	18	14	35	18	17	Lean Clay with Sand	CL

σ'v_o	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
7771	16000	15000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.01
Total Unit Weight (pcf)	132.23
Degree of Saturation (%)	98.90
Void Ratio (e0)	0.470

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
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Axial strain versus vertical effective stress for H-1vw-20 PS-23 CRS Consolidation

Job Number: 19501-27 5/25/2021

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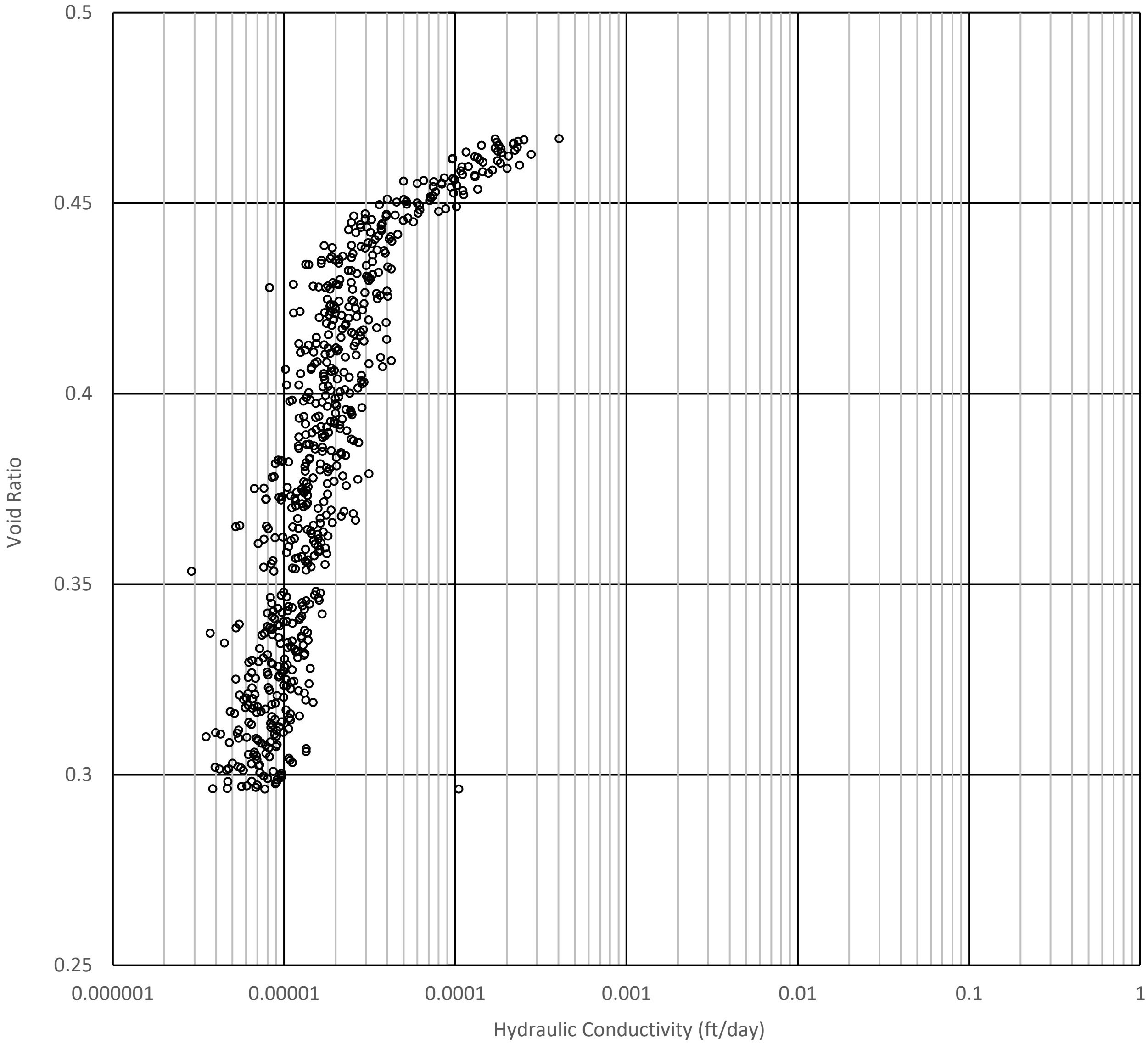
Figure
C-1-3



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
76.7	18	14	35	18	17	Lean Clay with Sand	CL

σ'v₀	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
7771	16000	15000
Sample Quality Designation		
Terzaghi et al. (1996)	**Lunne et al. (1997)**	
C	Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.01
Total Unit Weight (pcf)	132.23
Degree of Saturation (%)	98.90
Void Ratio (e0)	0.470

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
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Void ratio versus logarithm of hydraulic conductivity H-1vw-20 PS-23 CRS Consolidation

Job Number: 19501-27 5/25/2021

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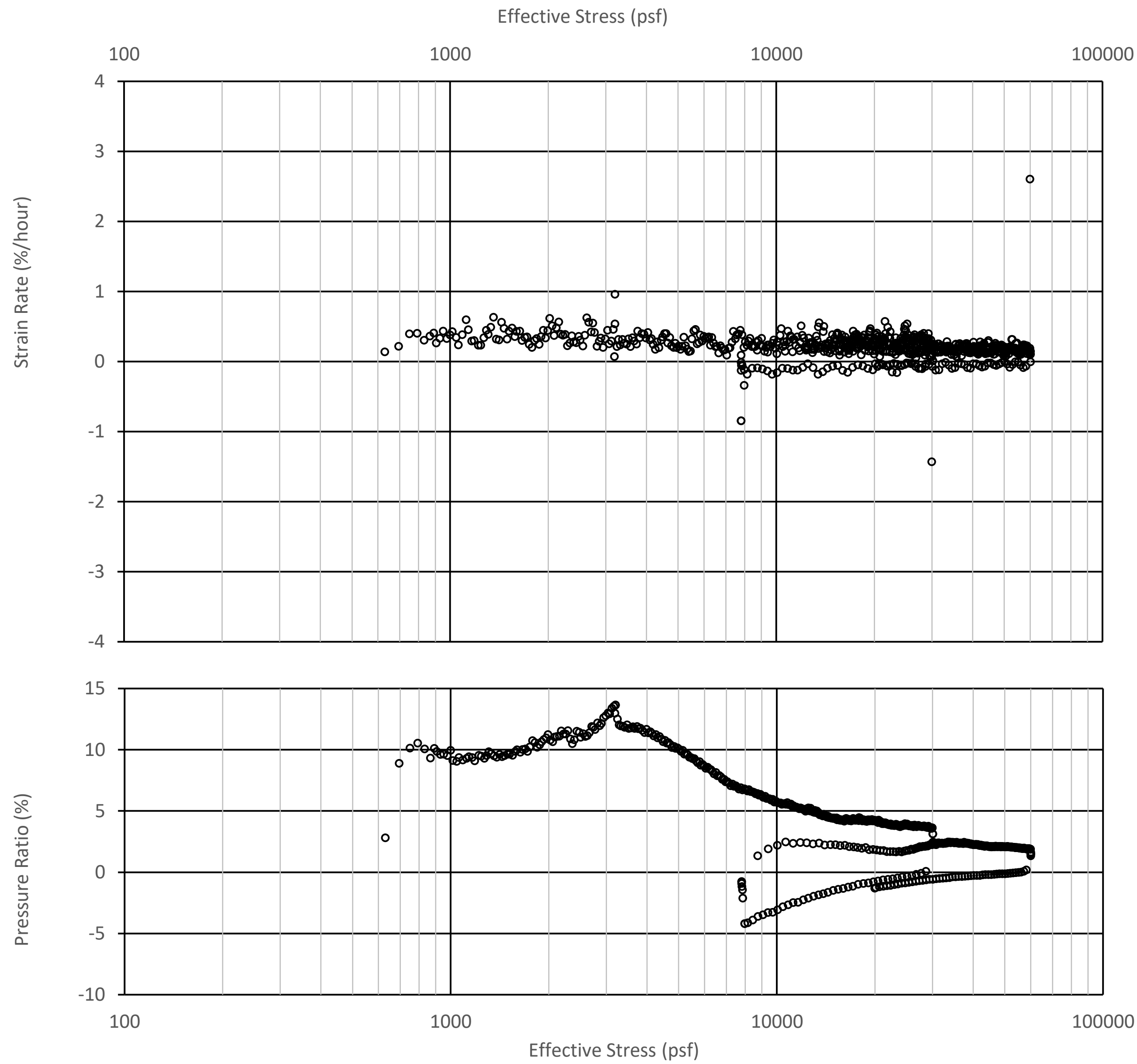
Figure
C-1-4



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
76.7	18	14	35	18	17	Lean Clay with Sand	CL

σ'_{v_o}	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
7771	16000	15000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.01
Total Unit Weight (pcf)	132.23
Degree of Saturation (%)	98.90
Void Ratio (e0)	0.470

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for H-1vw-20 PS-23 CRS Consolidation

Job Number: 19501-27 5/25/2021

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Figure
C-1-5

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Post-Test Specimen Photograph

19501-27 | 5/25/2021

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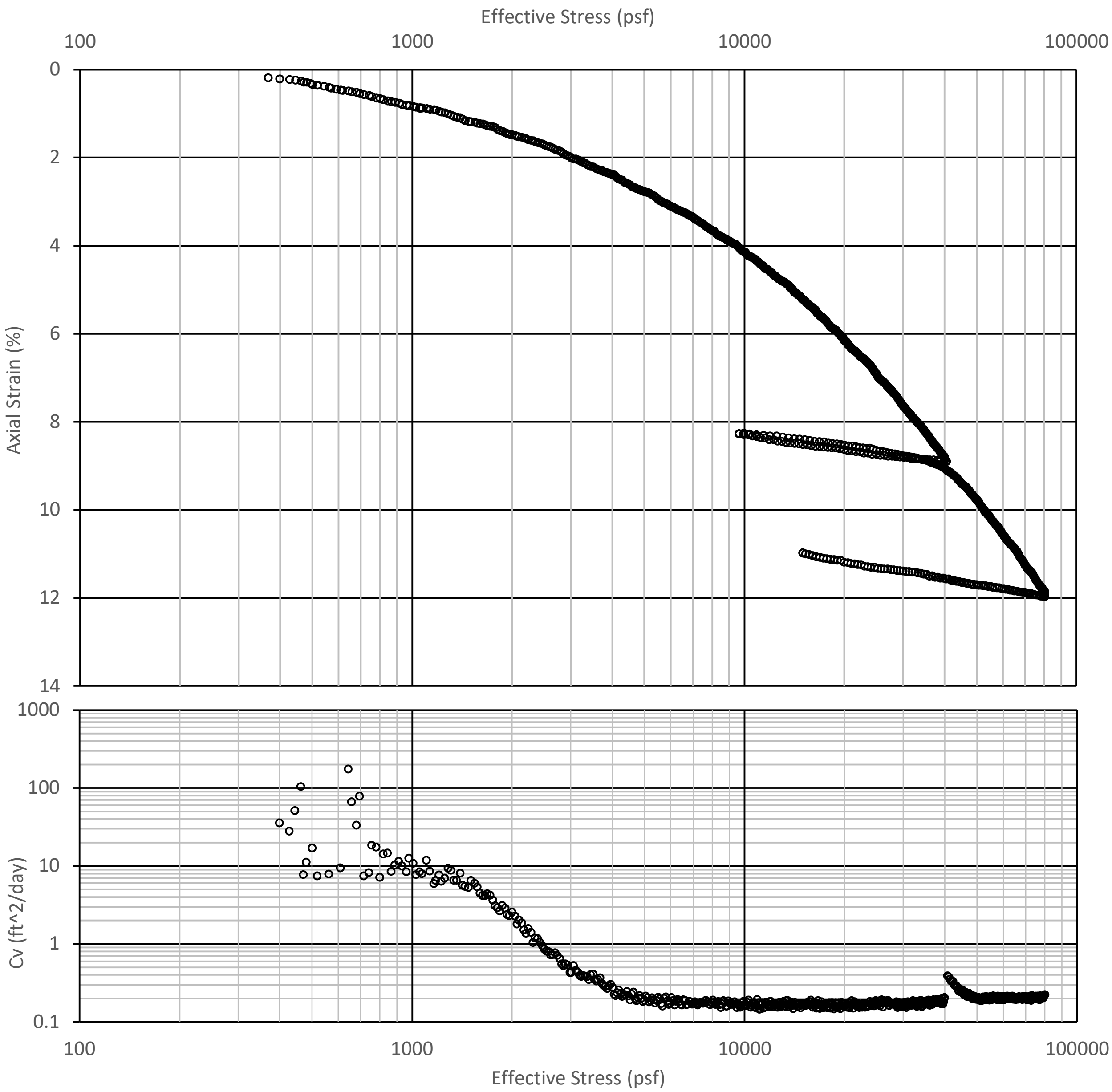
Figure
C-1-6



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
10.5	16	12	31	15	16	Lean Clay	CL

σ'_{v_o}	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
1320	10500	10000
Sample Quality Designation		
Terzaghi et al. (1996)	**Lunne et al. (1997)**	
B	Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.11
Total Unit Weight (pcf)	134.39
Degree of Saturation (%)	98.17
Void Ratio (e0)	0.423

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-6 CRS Consolidation

Job Number: 19501-27 6/2/2021

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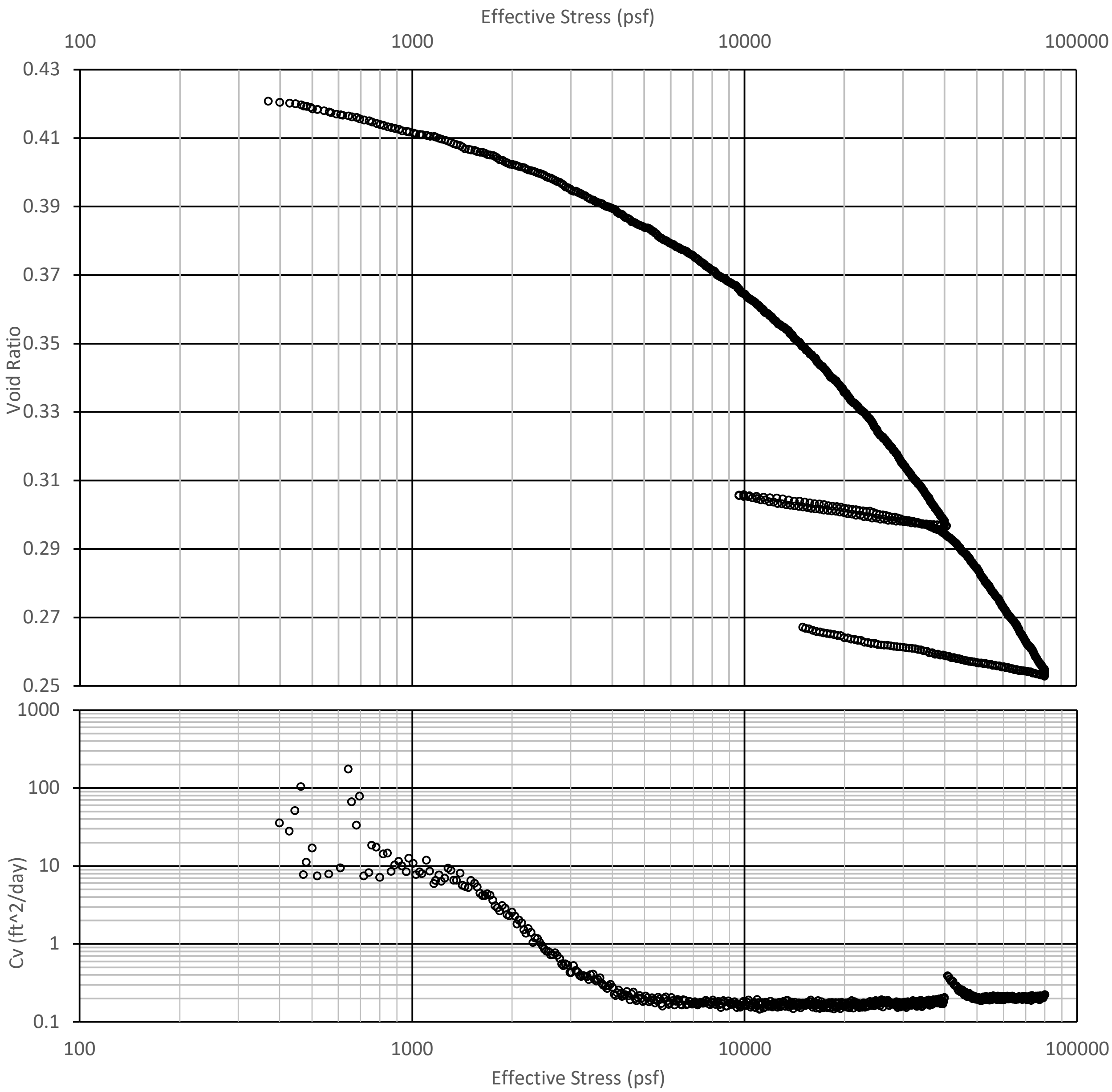
Figure
C-2-1



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
10.5	16	12	31	15	16	Lean Clay	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
1320	10500	10000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.11
Total Unit Weight (pcf)	134.39
Degree of Saturation (%)	98.17
Void Ratio (e0)	0.423

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-6 CRS Consolidation

Job Number: 19501-27 6/2/2021

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Figure
C-2-2

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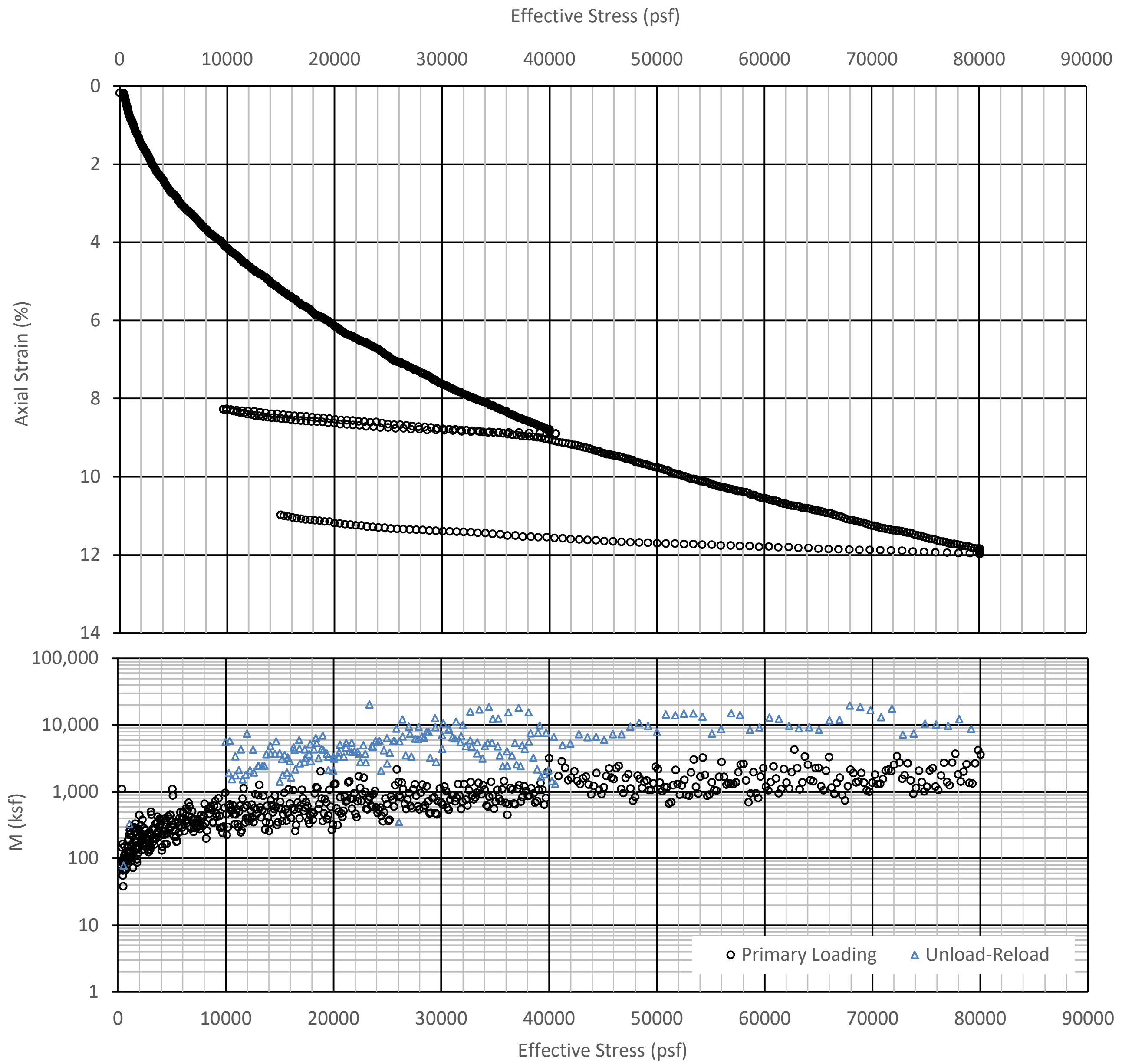
060221 DRAFT.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
10.5	16	12	31	15	16	Lean Clay	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
1320	10500	10000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.11
Total Unit Weight (pcf)	134.39
Degree of Saturation (%)	98.17
Void Ratio (e0)	0.423

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain versus vertical effective stress for H-4si-21 PS-6 CRS Consolidation

Job Number: 19501-27

6/2/2021

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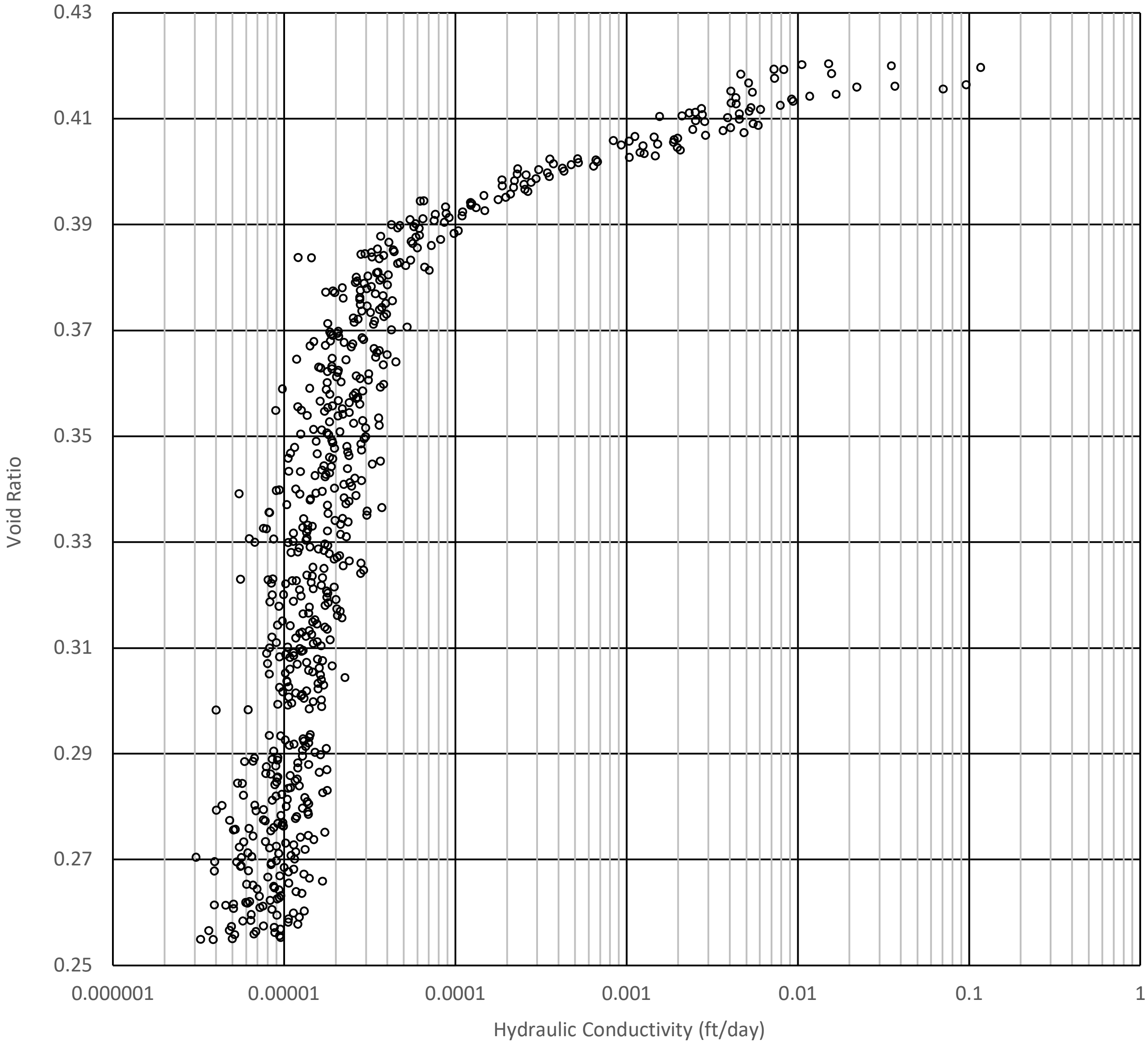
Figure
C-2-3



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
10.5	16	12	31	15	16	Lean Clay	CL

σ'vo	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
1320	10500	10000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.11
Total Unit Weight (pcf)	134.39
Degree of Saturation (%)	98.17
Void Ratio (e0)	0.423

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio versus logarithm of hydraulic conductivity H-4si-21 PS-6 CRS Consolidation

Job Number: 19501-27 6/2/2021

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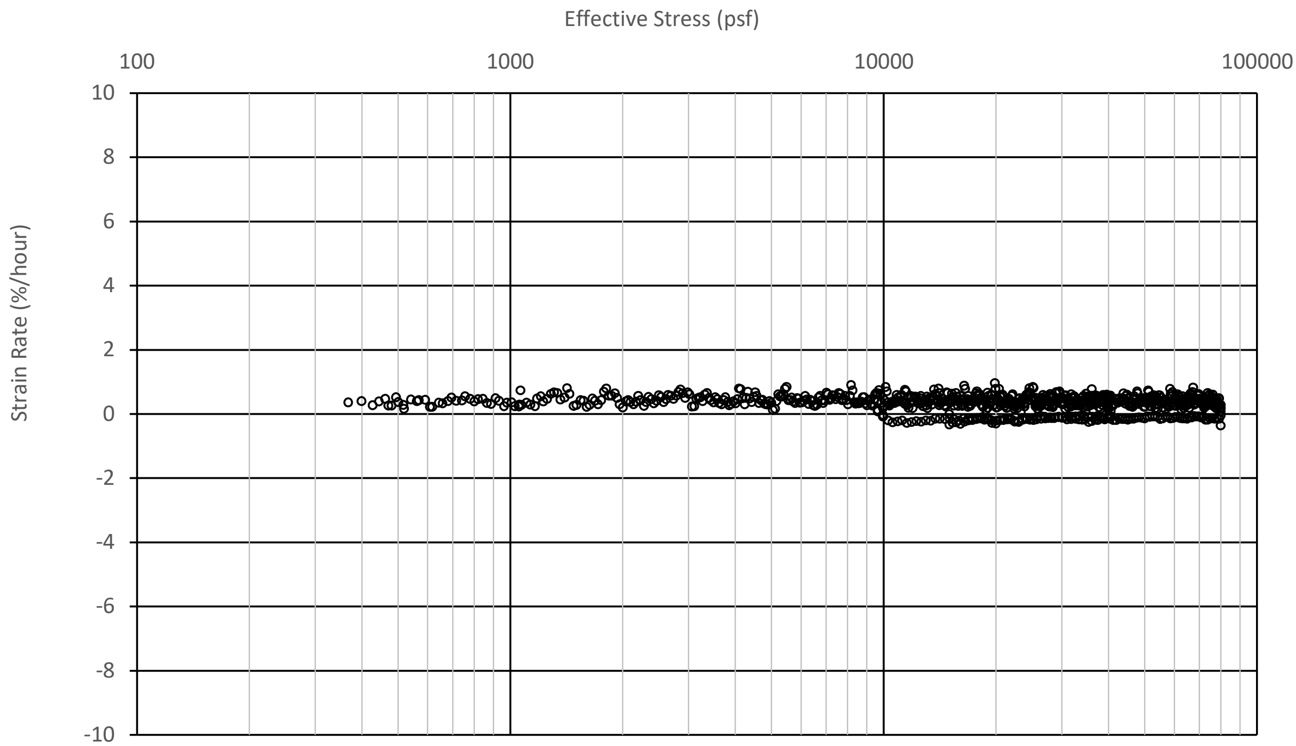
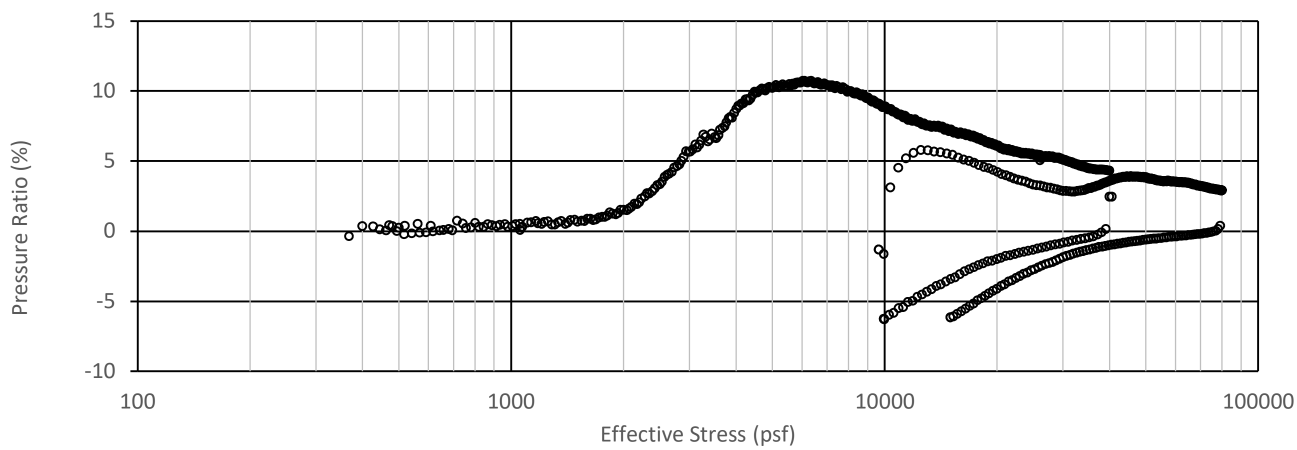
Figure
C-2-4





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
10.5	16	12	31	15	16	Lean Clay	CL

σ'vo	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
1320	10500	10000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.11
Total Unit Weight (pcf)	134.39
Degree of Saturation (%)	98.17
Void Ratio (e0)	0.423

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-6 CRS Consolidation

Job Number: 19501-27 6/2/2021

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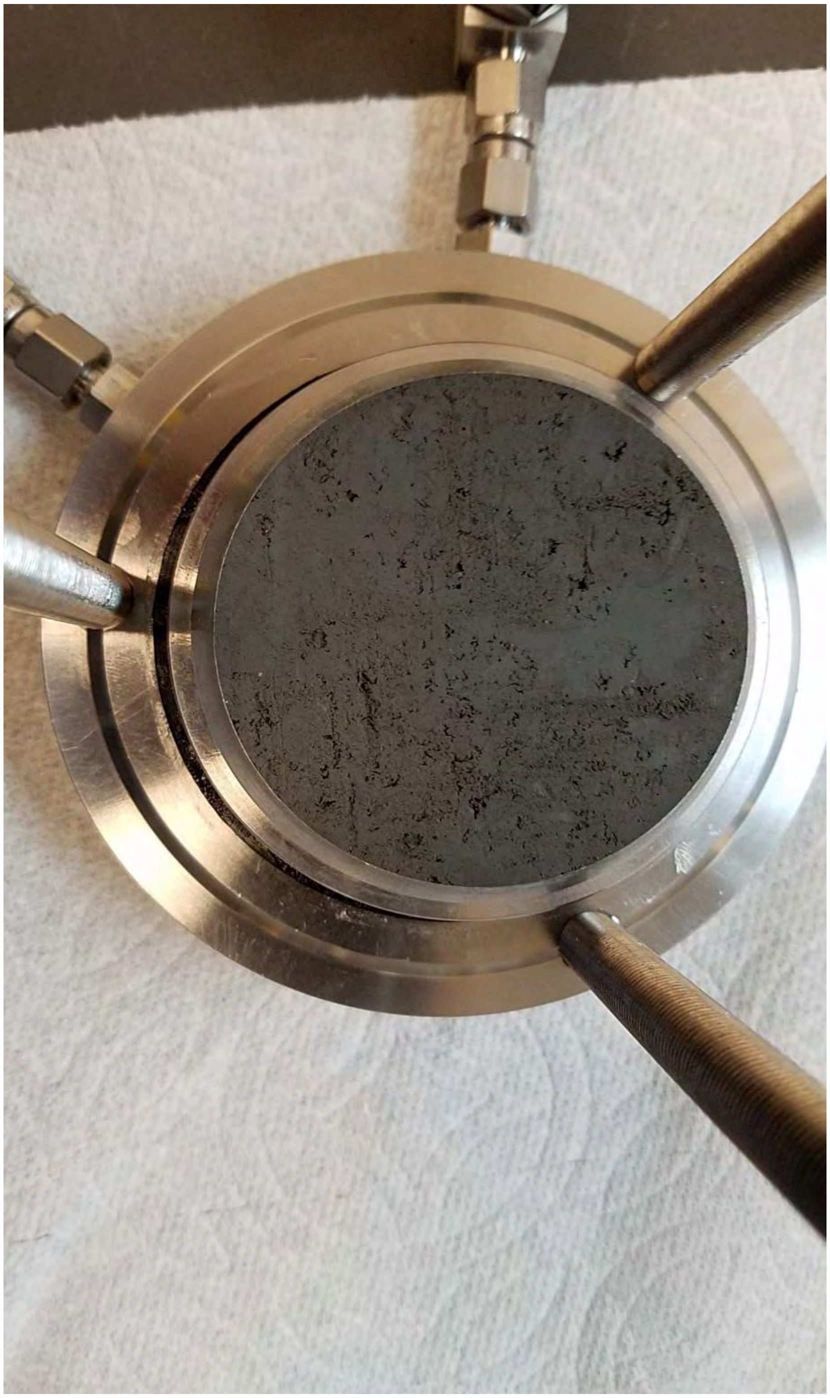
Figure
C-2-5



SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Pre-Test Photograph

19501-27 6/2/2021

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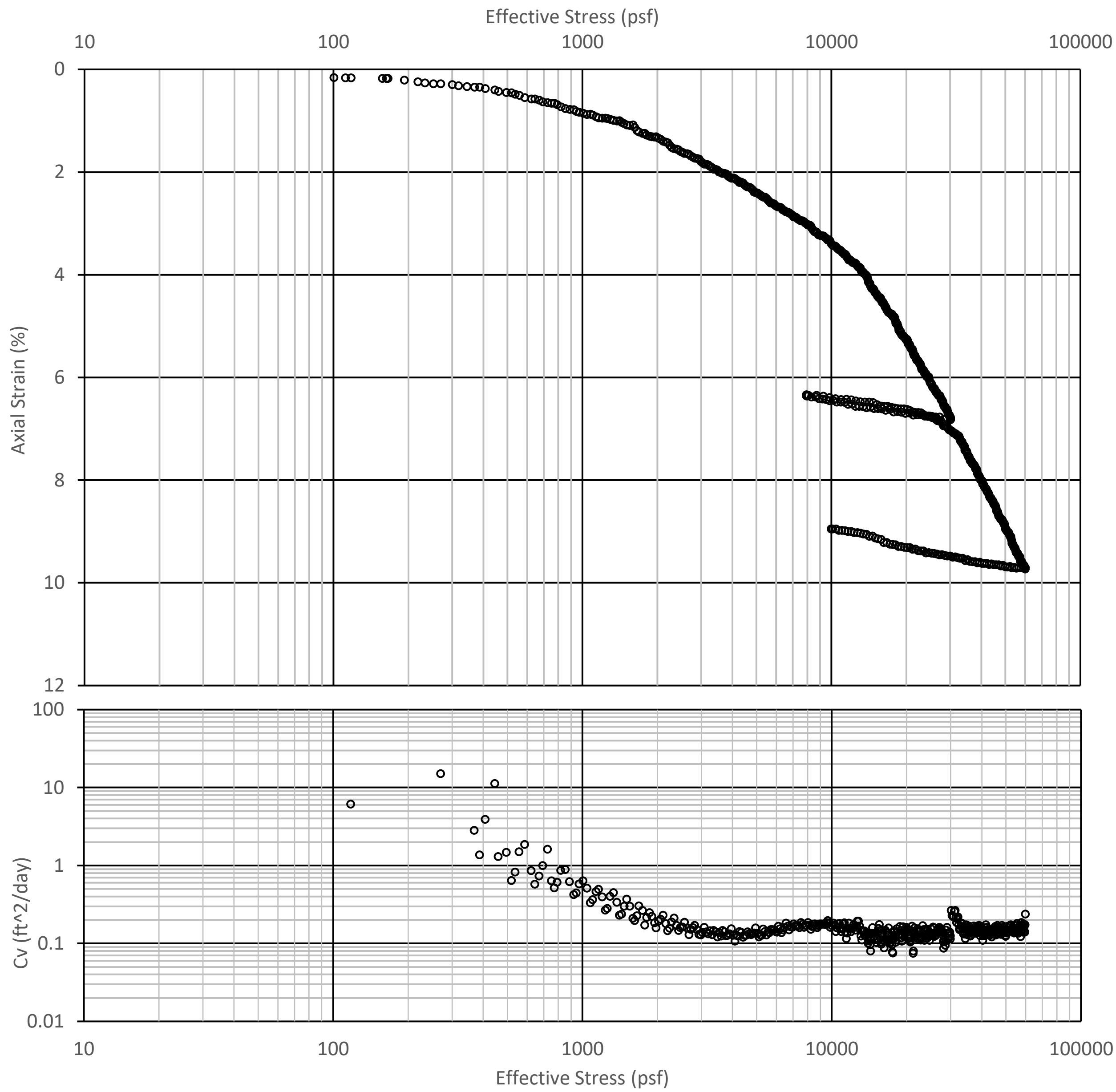
Figure

C-2-6



Depth	W.C. (%)		Atterberg Limits				
(ft)	Before	After	LL	PL	PI	Description	USCS
20.2	15	13	26	13	13	Lean Clay	CL

σ'vₒ	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
2520	14000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Very good to excellent	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.16
Total Unit Weight (pcf)	135.49
Degree of Saturation (%)	99.18
Void Ratio (e0)	0.406

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-12 CRS Consolidation

Job Number: 15901-27 5/27/2021

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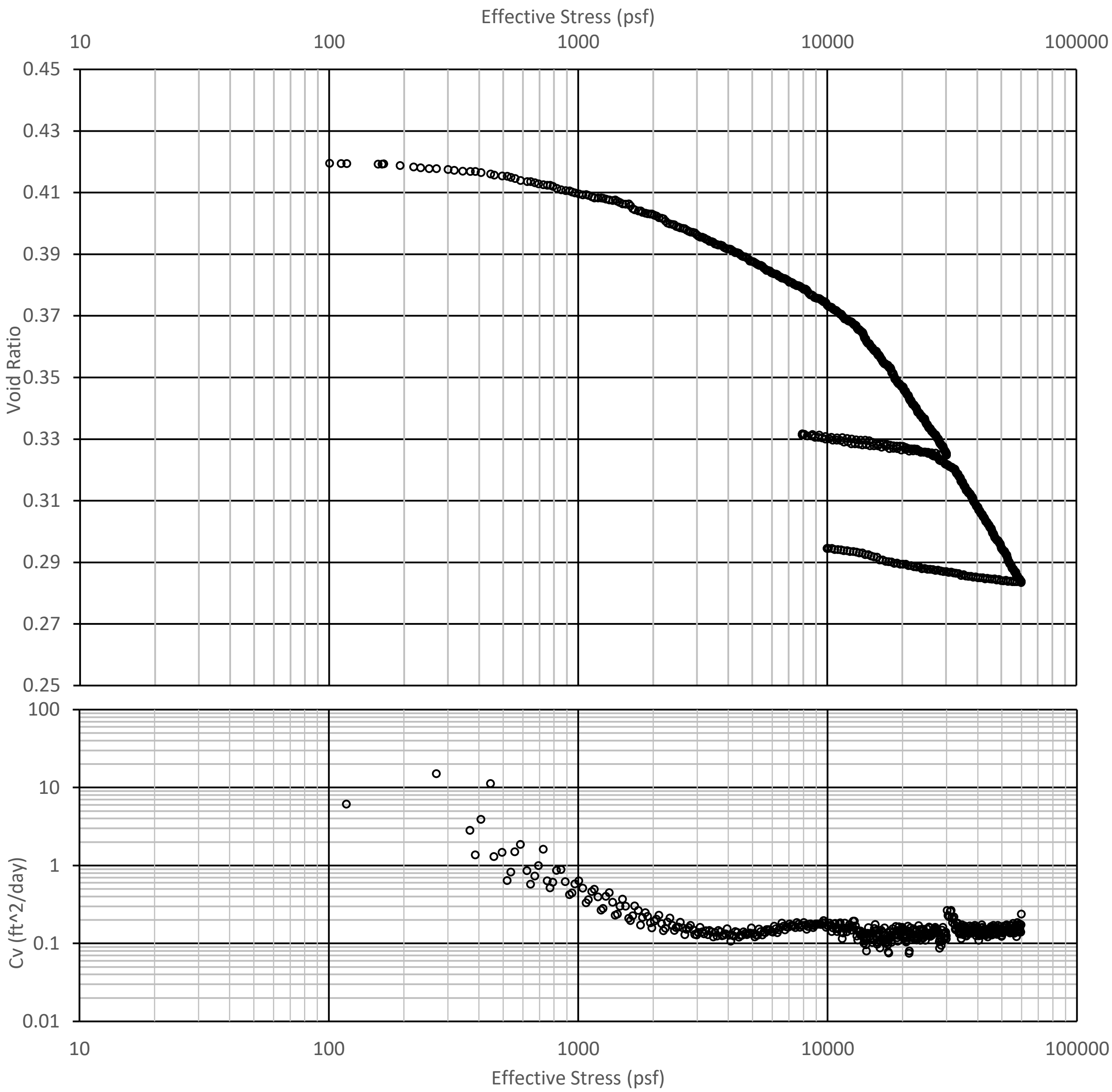
Figure
C-3-1



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
20.2	15	13	26	13	13	Lean Clay	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
2520	14000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Very good to excellent	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.16
Total Unit Weight (pcf)	135.49
Degree of Saturation (%)	99.18
Void Ratio (e0)	0.406

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-12 CRS Consolidation

Job Number: 15901-27 5/27/2021

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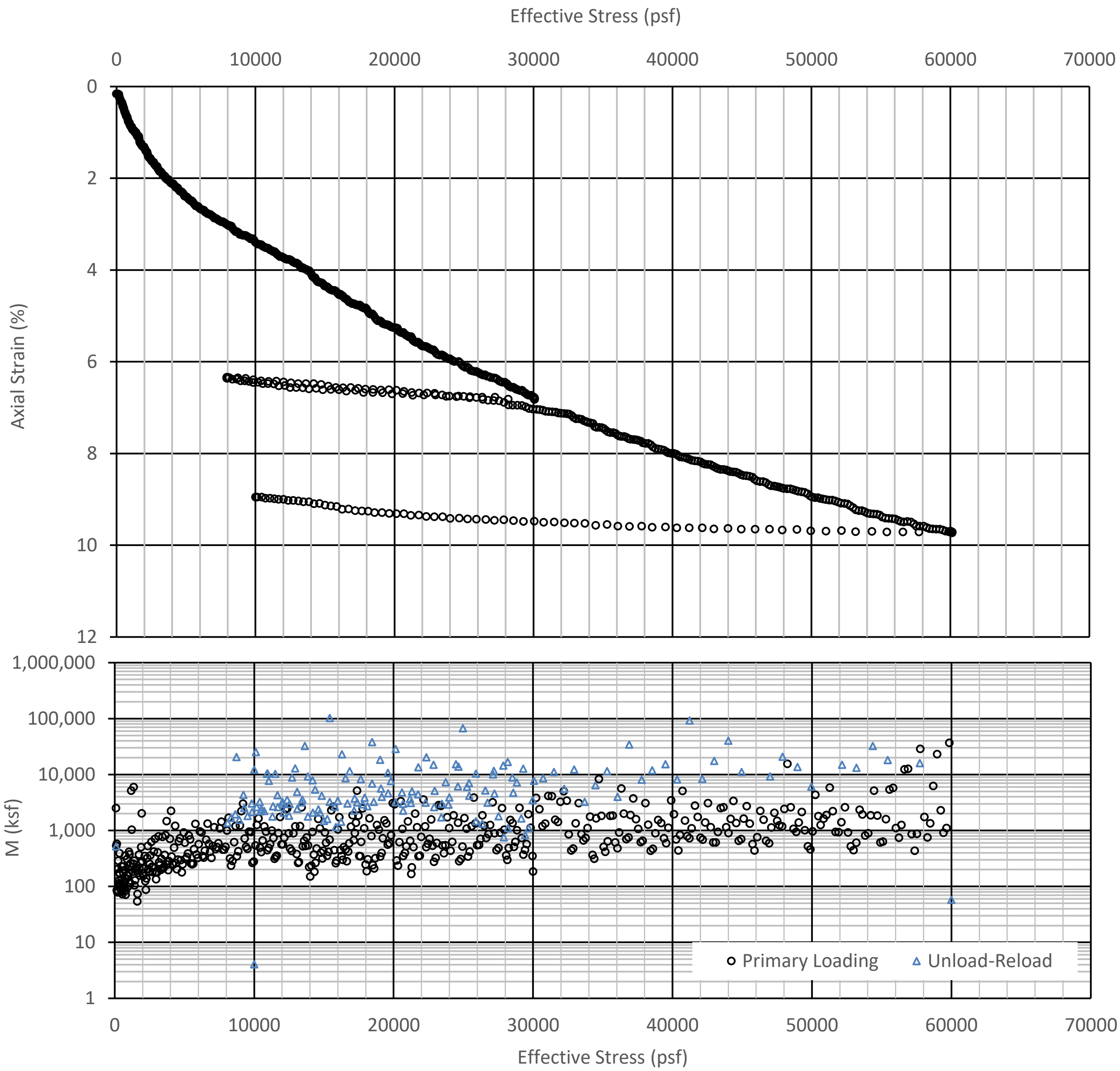
Figure
C-3-2

ML 7/18/2022 \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\Deliverables In-Basket\FINAL Report\Attachments\Appendices\Appendix B & C - Laboratory Data\Appendix C\CRS\19501-27 H-4si-21 PS-12 CRS2 052721 DRAFT.xlsm



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
20.2	15	13	26	13	13	Lean Clay	CL

σ'_{v_o}	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
2520	14000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Very good to excellent	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.16
Total Unit Weight (pcf)	135.49
Degree of Saturation (%)	99.18
Void Ratio (e0)	0.406

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

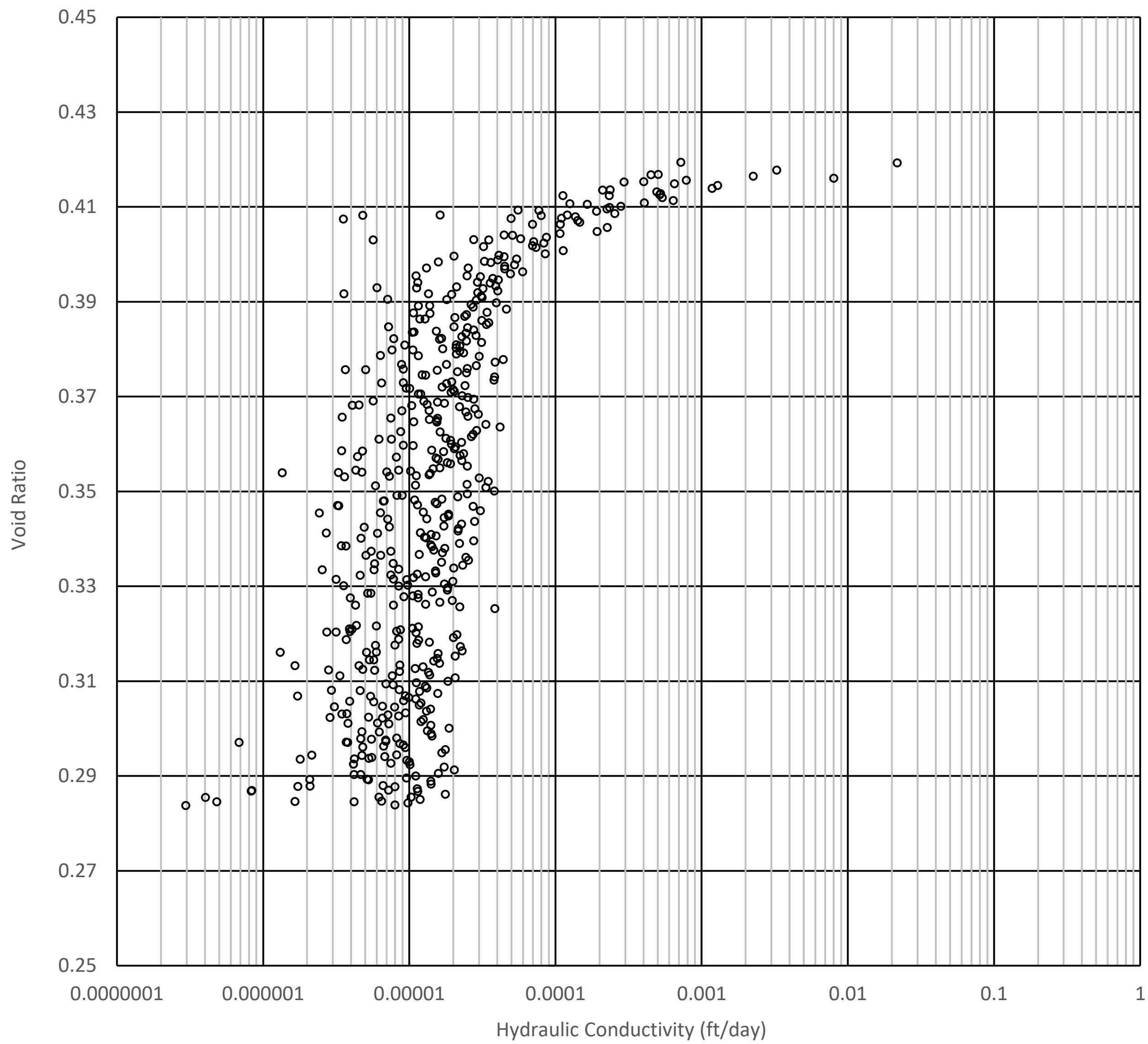
SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain versus vertical effective stress for H-4si-21 PS-12 CRS Consolidation

Job Number: 15901-27 5/27/2021

HARTCROWSER
A division of Haley & Aldrich

Figure
C-3-3



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
20.2	15	13	26	13	13	Lean Clay	CL

σ'vₒ	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
2520	14000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Very good to excellent	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.16
Total Unit Weight (pcf)	135.49
Degree of Saturation (%)	99.18
Void Ratio (e0)	0.406

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

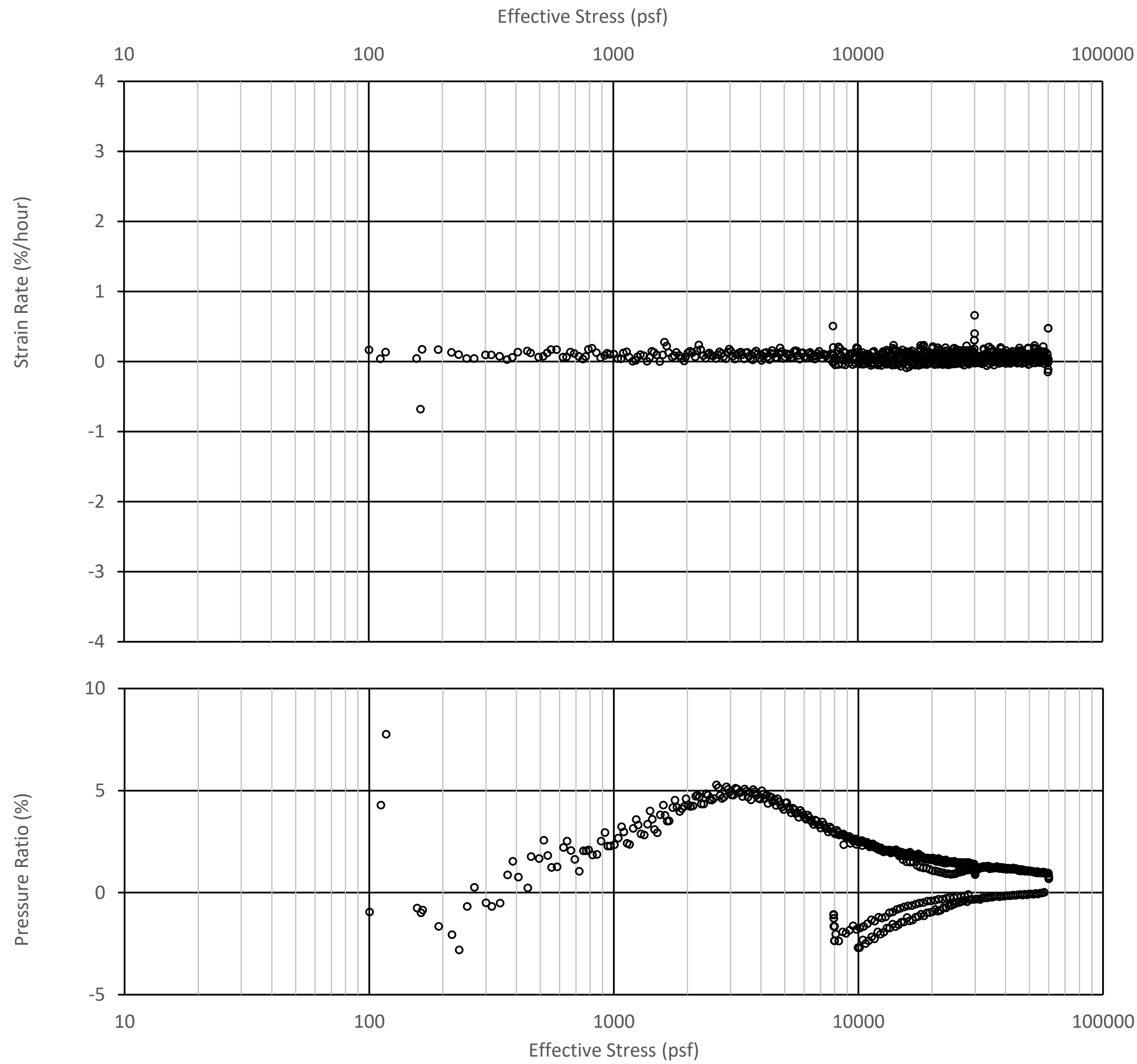
SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio versus logarithm of hydraulic conductivity H-4si-21 PS-12 CRS Consolidation

Job Number: 15901-27 5/27/2021

HARTCROWSER
A division of Haley & Aldrich

Figure
C-3-4



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
20.2	15	13	26	13	13	Lean Clay	CL

σ'_{vo}	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
2520	14000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
B	Very good to excellent	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.16
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Degree of Saturation (%)	99.18
Void Ratio (e0)	0.406

Sample Preparation and Comments:
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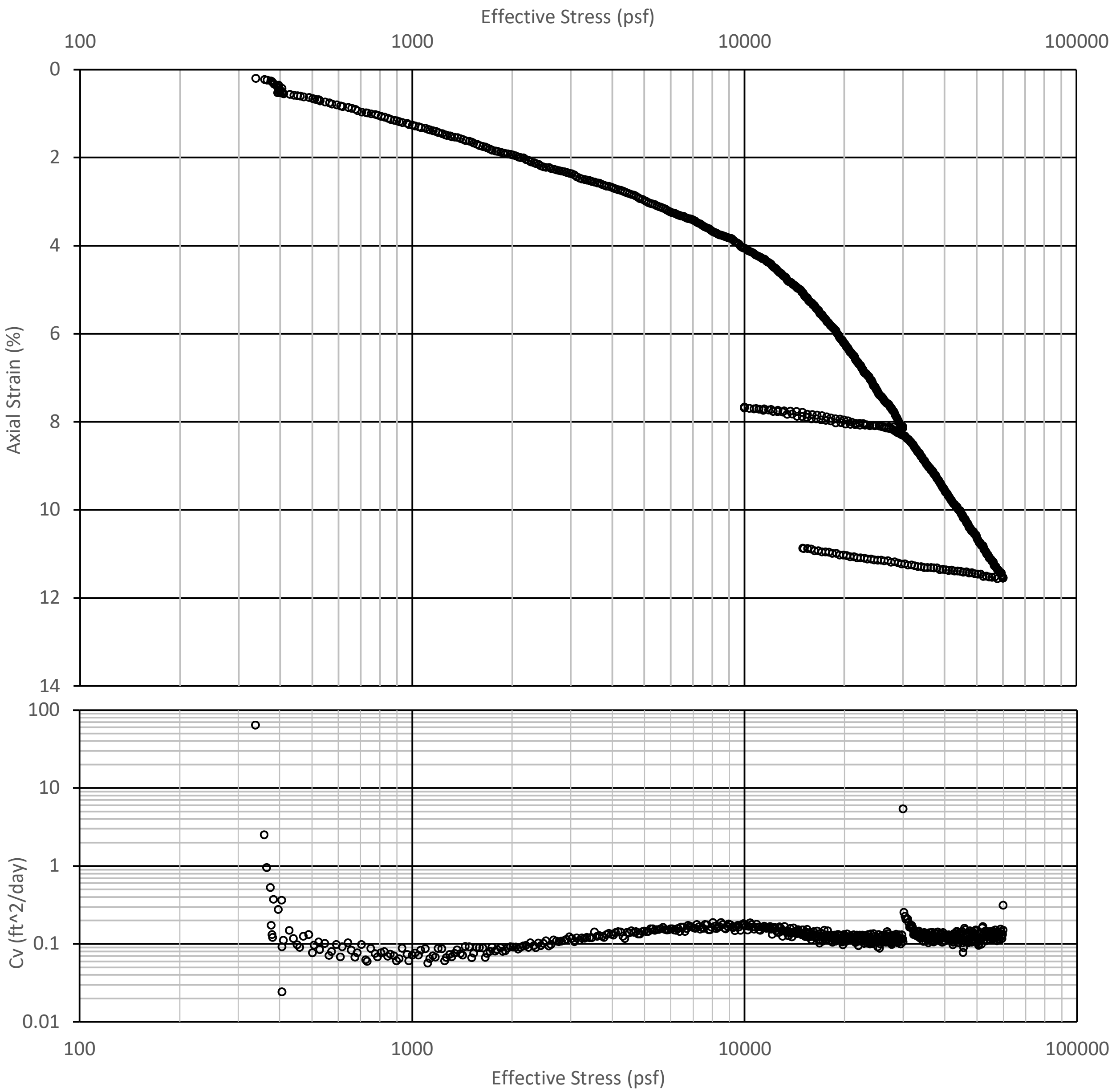
SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-12 CRS Consolidation

Job Number: 15901-27 5/27/2021

HARTCROWSER A division of Haley & Aldrich

Figure
C-3-5



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	17	14	28	14	14	Lean Clay	CL

σ'vo	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
3720	13000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	**Lunne et al. (1997)**	
C	#N/A	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.04
Total Unit Weight (pcf)	132.83
Degree of Saturation (%)	97.84
Void Ratio (e0)	0.453

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

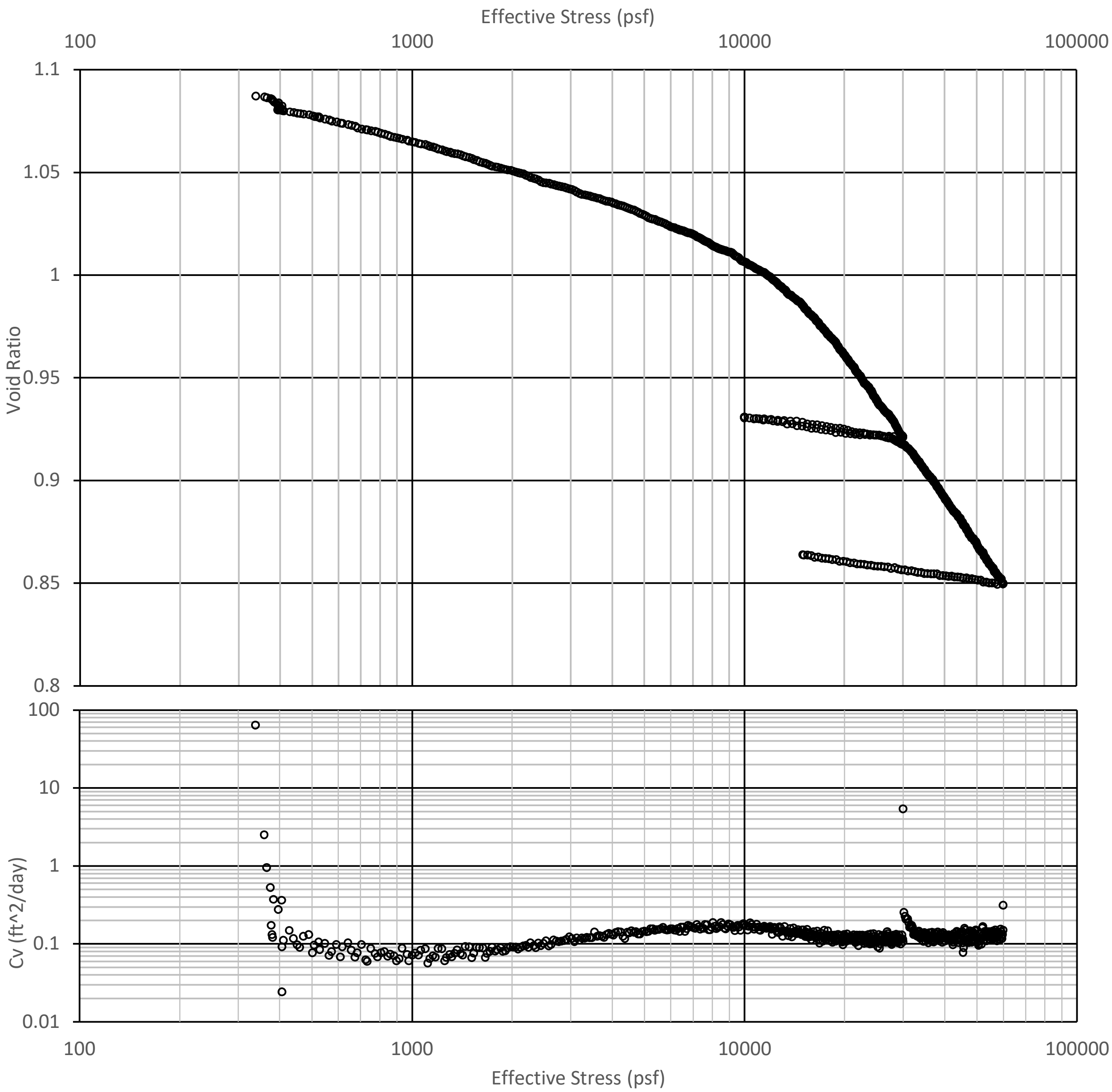
SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-18 CRS Consolidation

Job Number: 19501-27 6/11/2021

HARTCROWSER
A division of Haley & Aldrich

Figure
C-4-1



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	17	14	28	14	14	Lean Clay	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
3720	13000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	#N/A	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
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Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

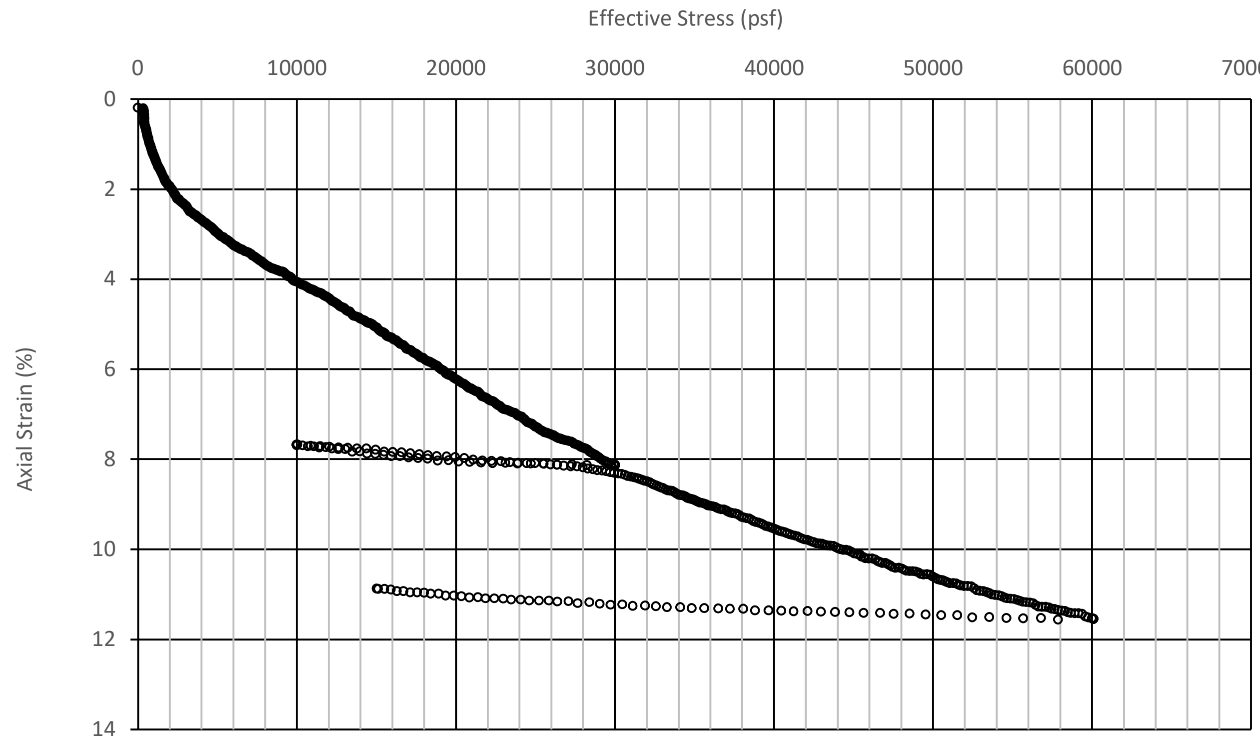
SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

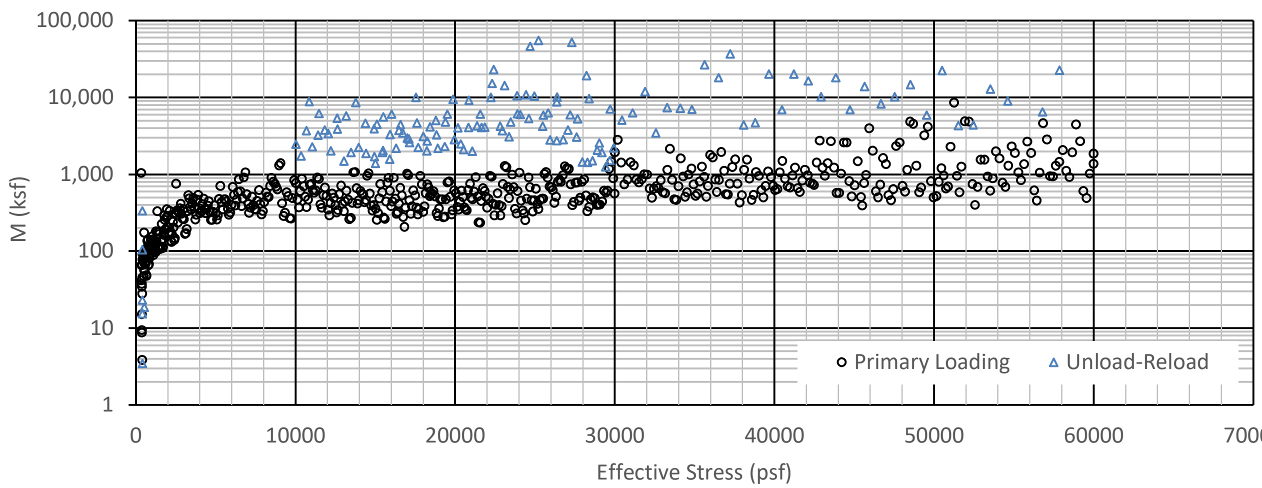
Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-18 CRS Consolidation

Job Number: 19501-27 6/11/2021

HARTCROWSER
A division of Haley & Aldrich

Figure
C-4-2





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	17	14	28	14	14	Lean Clay	CL

σ'v_o	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
3720	13000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	#N/A	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.04
Total Unit Weight (pcf)	132.83
Degree of Saturation (%)	97.84
Void Ratio (e0)	0.453

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain versus vertical effective stress for H-4si-21 PS-18 CRS Consolidation

Job Number: 19501-27

6/11/2021

HARTCROWSER
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Figure
C-4-3

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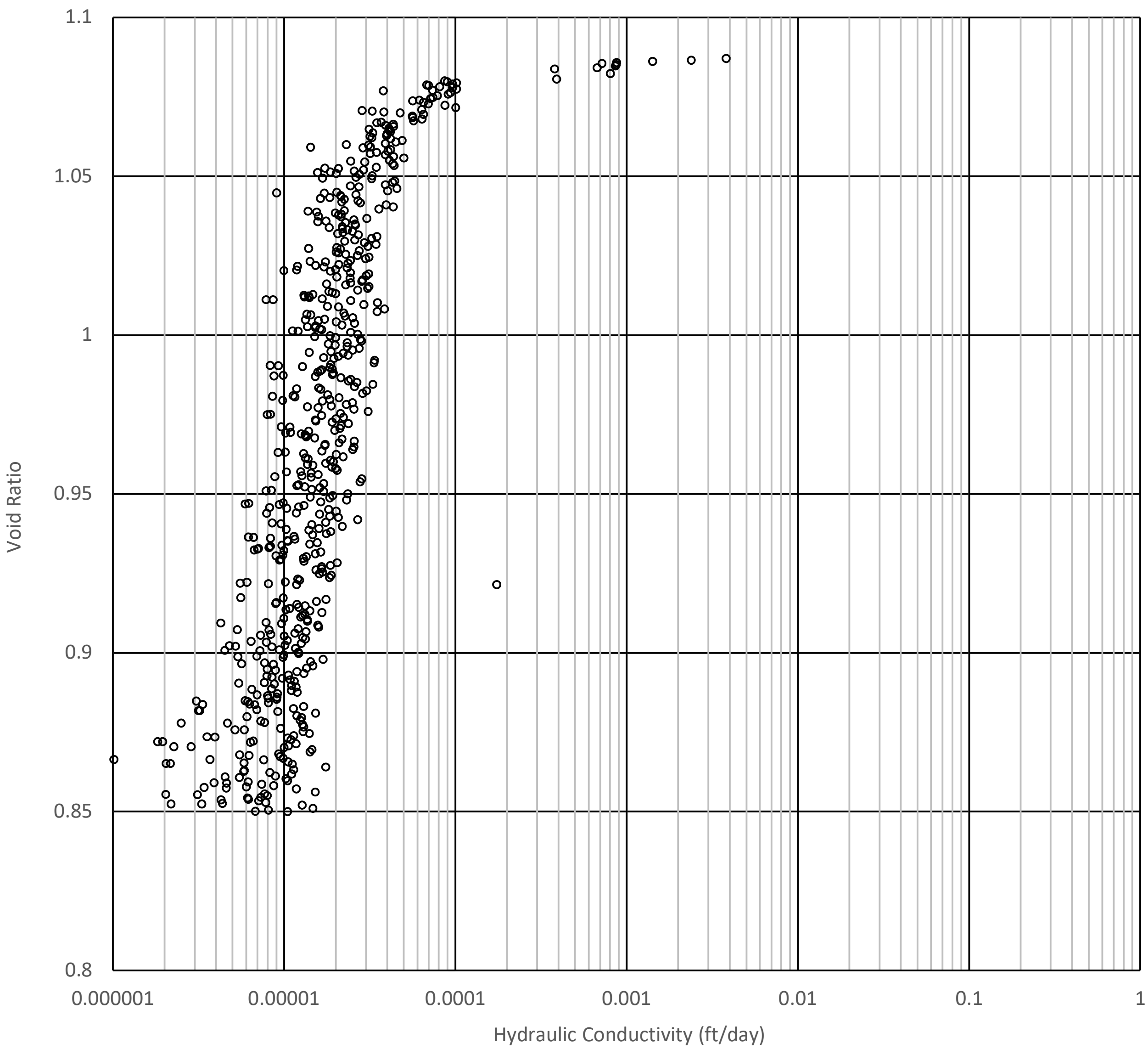

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	17	14	28	14	14	Lean Clay	CL

σ'_{v_o}	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
3720	13000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	#N/A	

Initial Specimen Properties	
Height (inches)	1.00
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Total Unit Weight (pcf)	132.83
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Void Ratio (e0)	0.453

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Void ratio versus logarithm of hydraulic conductivity H-4si-21 PS-18 CRS Consolidation

Job Number: 19501-27 6/11/2021

HARTCROWSER
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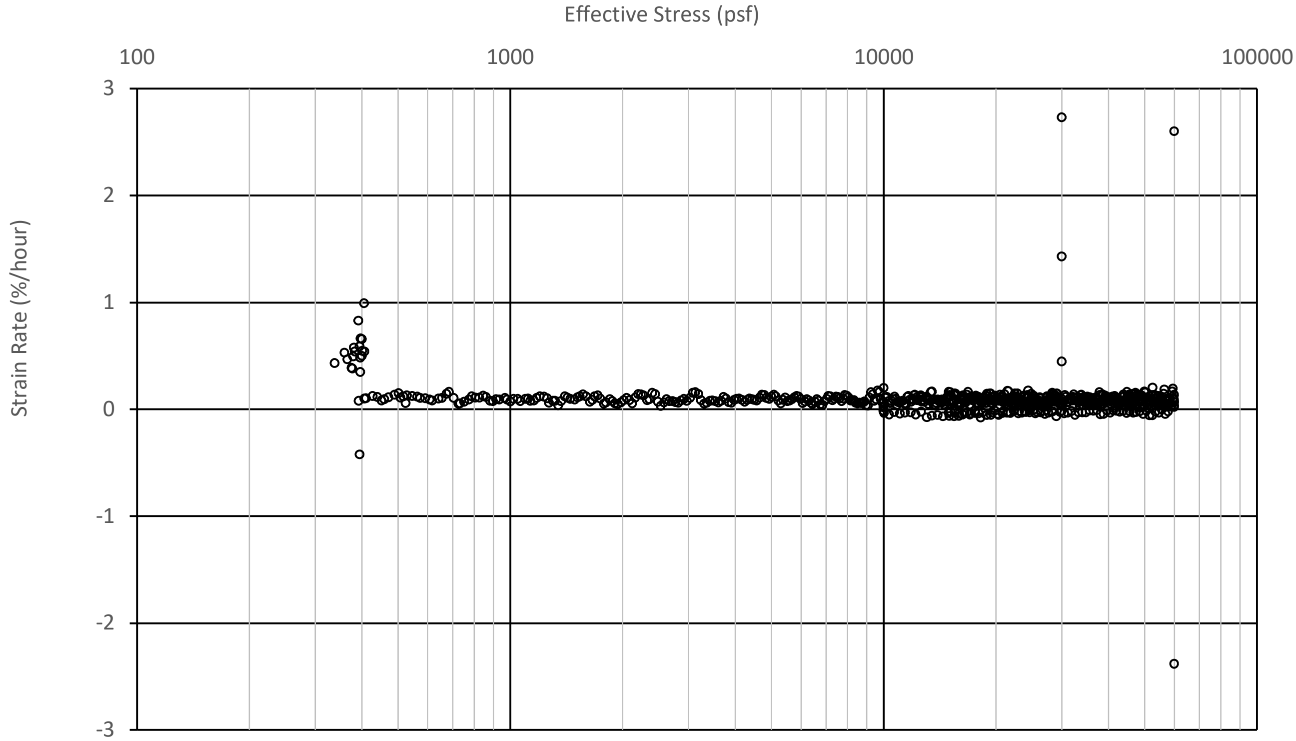
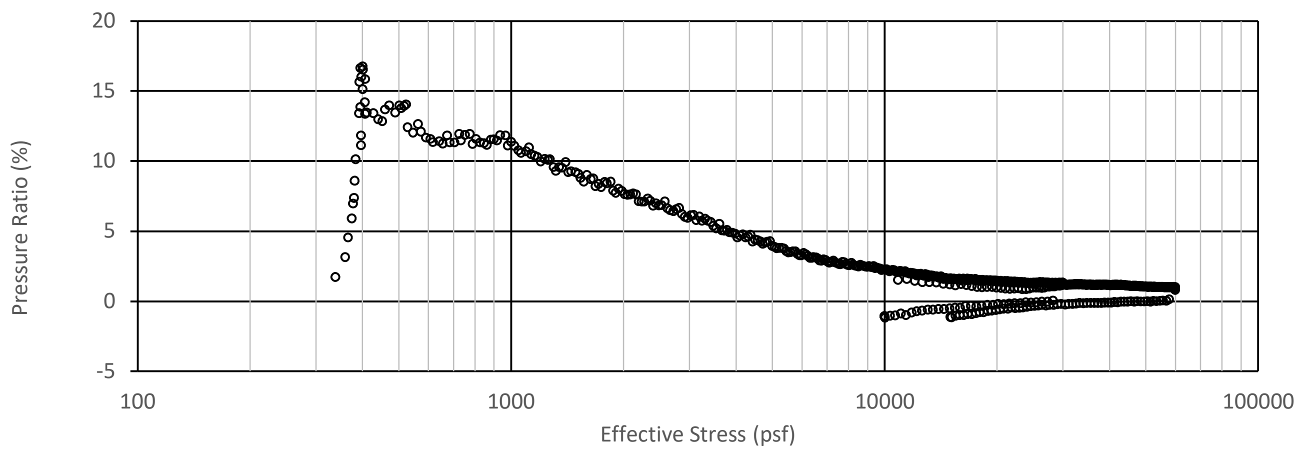
Figure
C-4-4





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	17	14	28	14	14	Lean Clay	CL

σ'v$_o$	Preconsolidation Pressure (psf)	
(psf)	Strain Energy	Casagrande
3720	13000	13000
Sample Quality Designation		
Terzaghi et al. (1996)	Lunne et al. (1997)	
C	#N/A	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	6.04
Total Unit Weight (pcf)	132.83
Degree of Saturation (%)	97.84
Void Ratio (e0)	0.453

Sample Preparation and Comments:
The specimen test was an intact soil sample which was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run with a room temperature between 71 and 73 degrees Farenheit.

SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for H-4si-21 PS-18 CRS Consolidation

Job Number: 19501-27 6/11/2021

HARTCROWSER
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Figure
C-4-5

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SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Bellingham, WA

Pre-Test Photograph

19501-27 6/11/2021

HARTCROWSER
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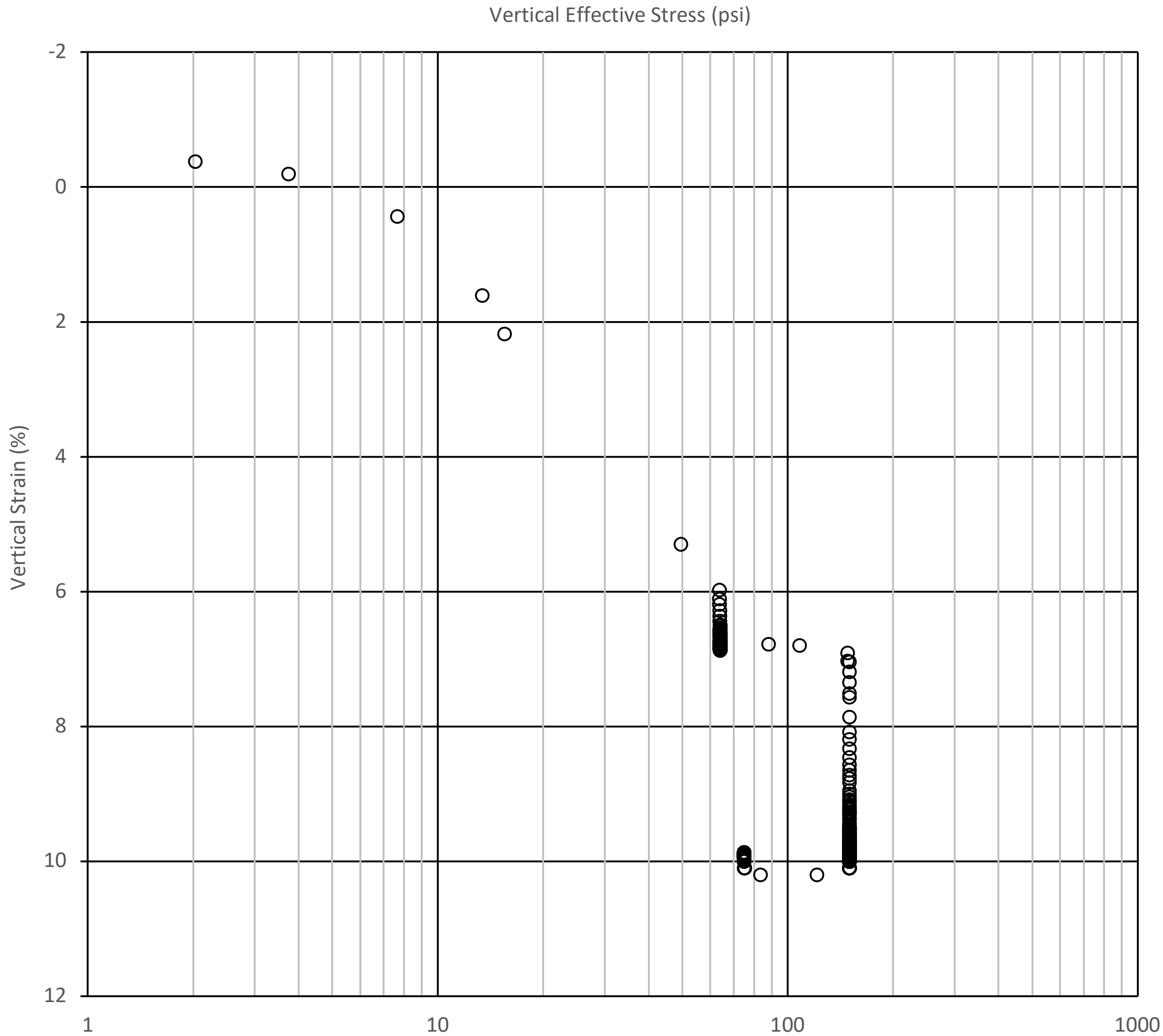
Figure
C-4-6



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
87	77	72	38	17	21	LEAN CLAY with SAND	CL

Partical-size Distribution		
% Gravel	% Sand	% Fines
NT	NT	76.3

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.25
Total Unit Weight (pcf)	93.57
Degree of Saturation (%)	98.68
Void Ratio (e_0)	1.970

Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

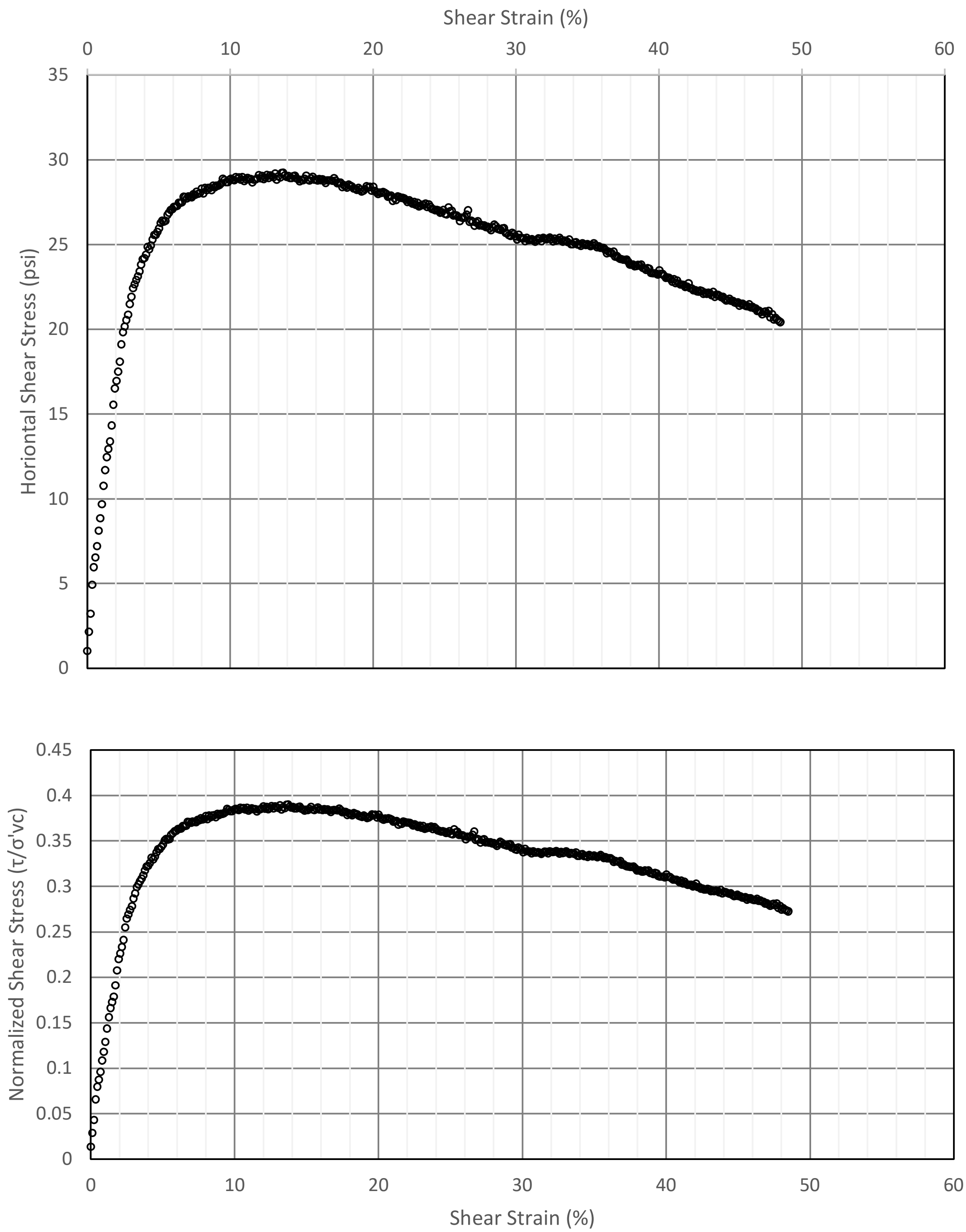
Squalicum FP
Bellingham, WA

Axial Strain Versus Logarithm of Vertical Effective Stress for H-2p-20 PS-29 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-5-1



Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bellingham, WA

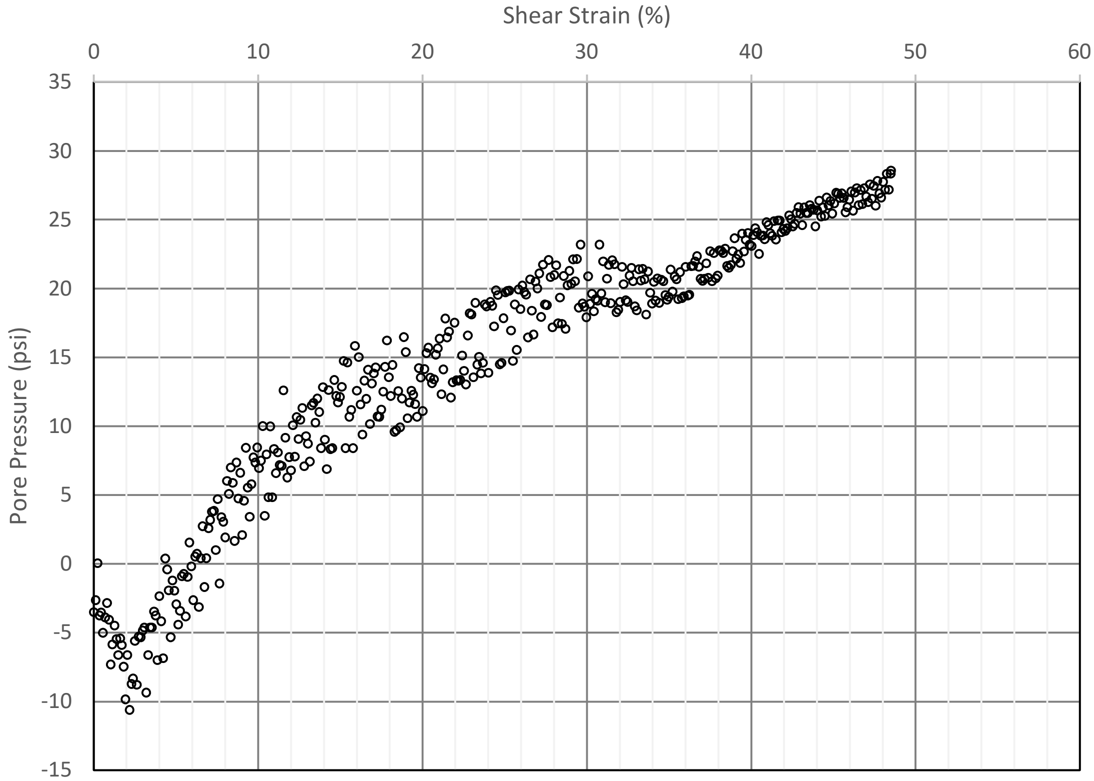
Horizontal Shear Stress and Normalized Shear Stress for H-2p-20 PS-29 Specimen#1 DSS

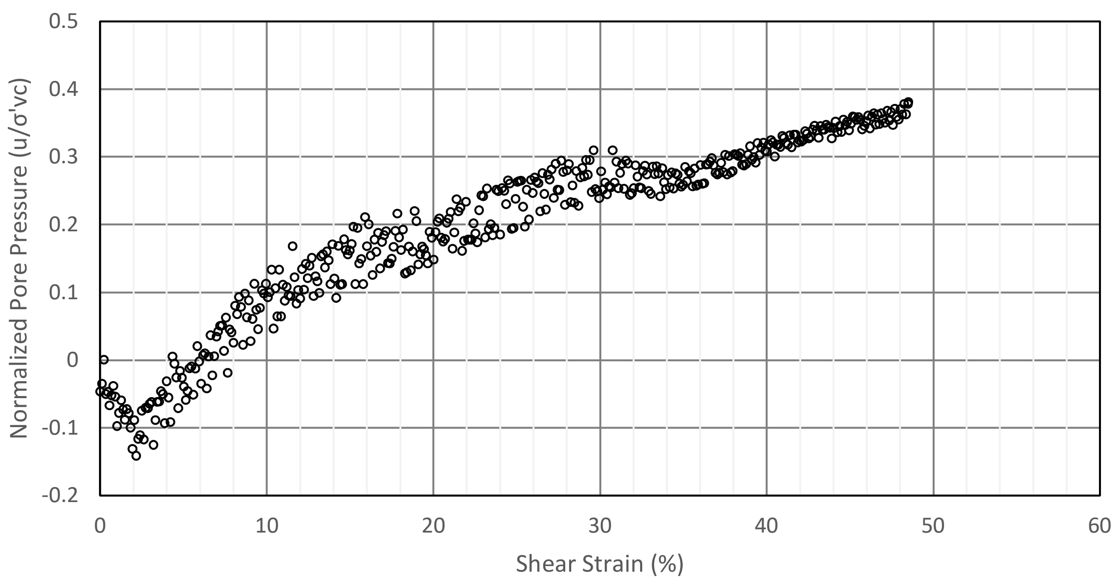
Job Number: 19501-27

06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-5-2





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

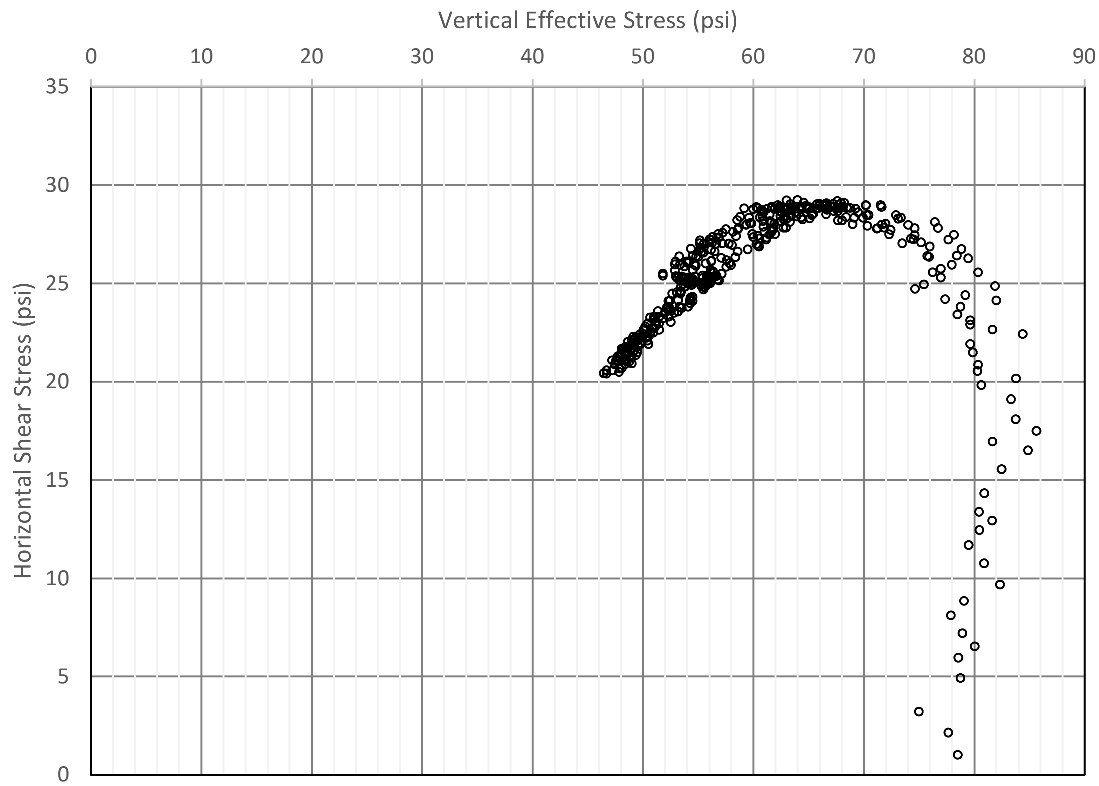
Squalicum FP
Bellingham, WA

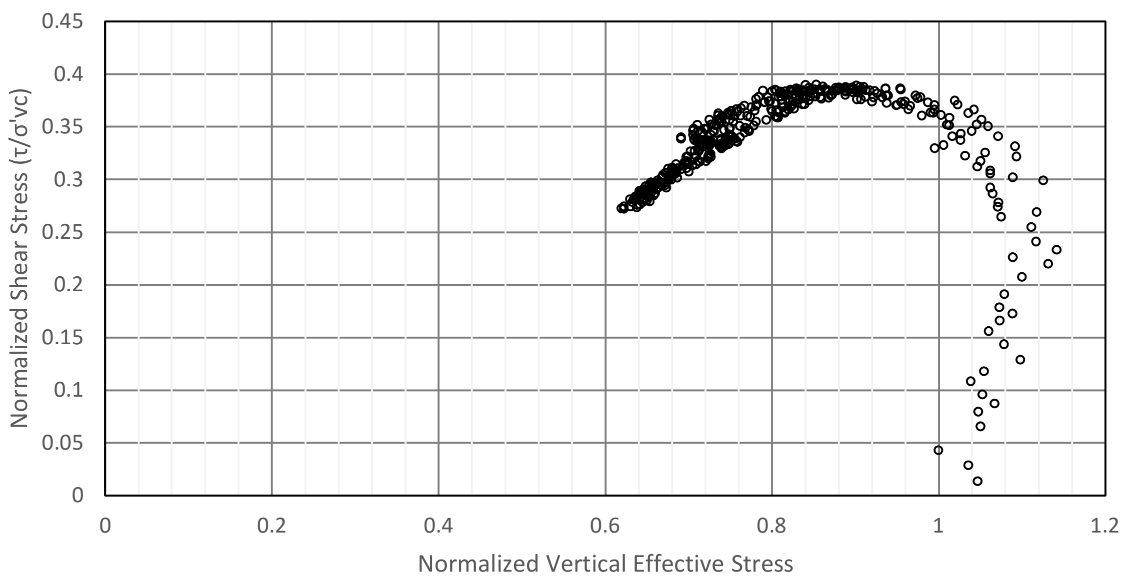
Pore Pressure and Normalized Pore Pressure Versus Shear Strain for H-2p-20 PS-29 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-5-3





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

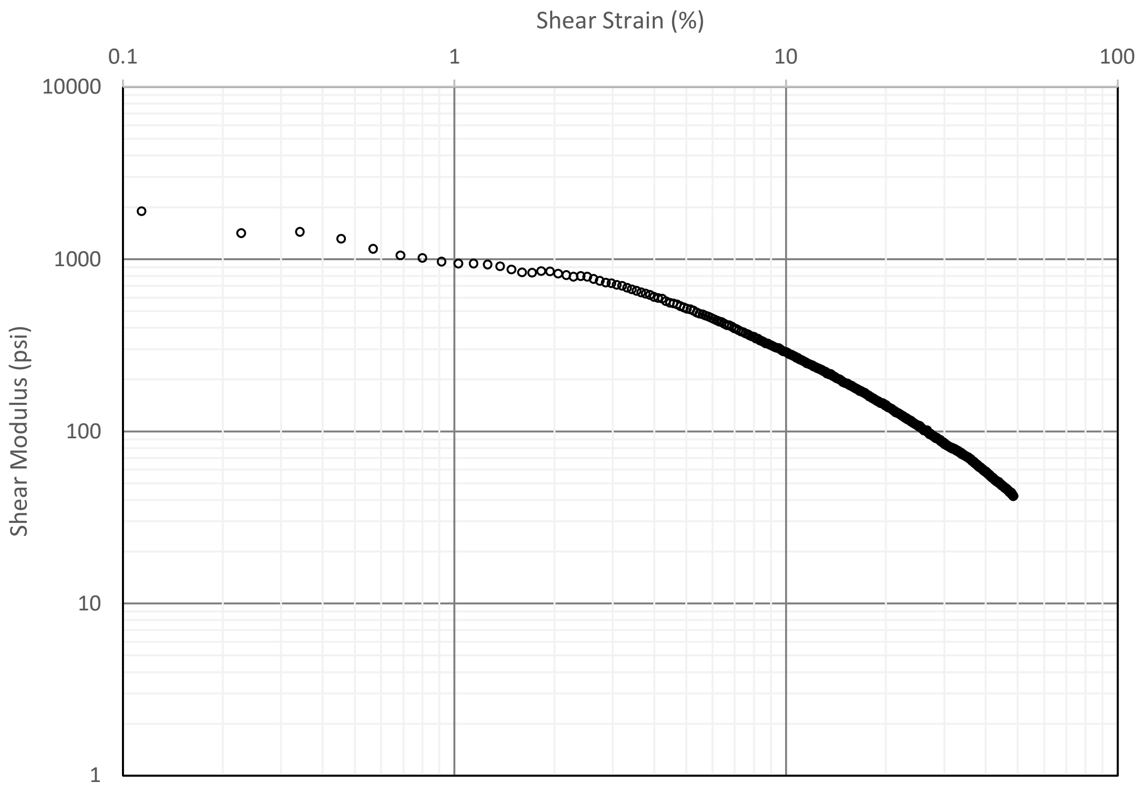
Squalicum FP
Bellingham, WA

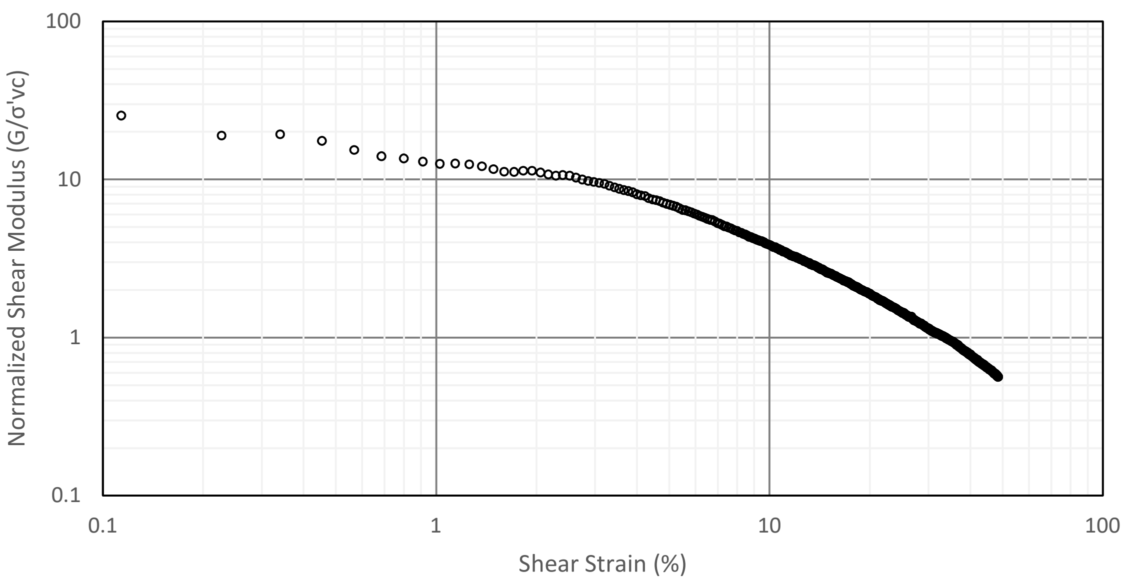
Horizontal and Normalized Shear Stress Versus Vertical and Normalized Vertical Effective Stress for H-2p-20 PS-29 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-5-4





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bellingham, WA

Shear Modulus and Normalized Shear Modulus Versus Shear Strain for H-2p-20 PS-29 Specimen#1 DSS

Job Number: 19501-27

06/21

HARTCROWSER
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Figure

C-5-5

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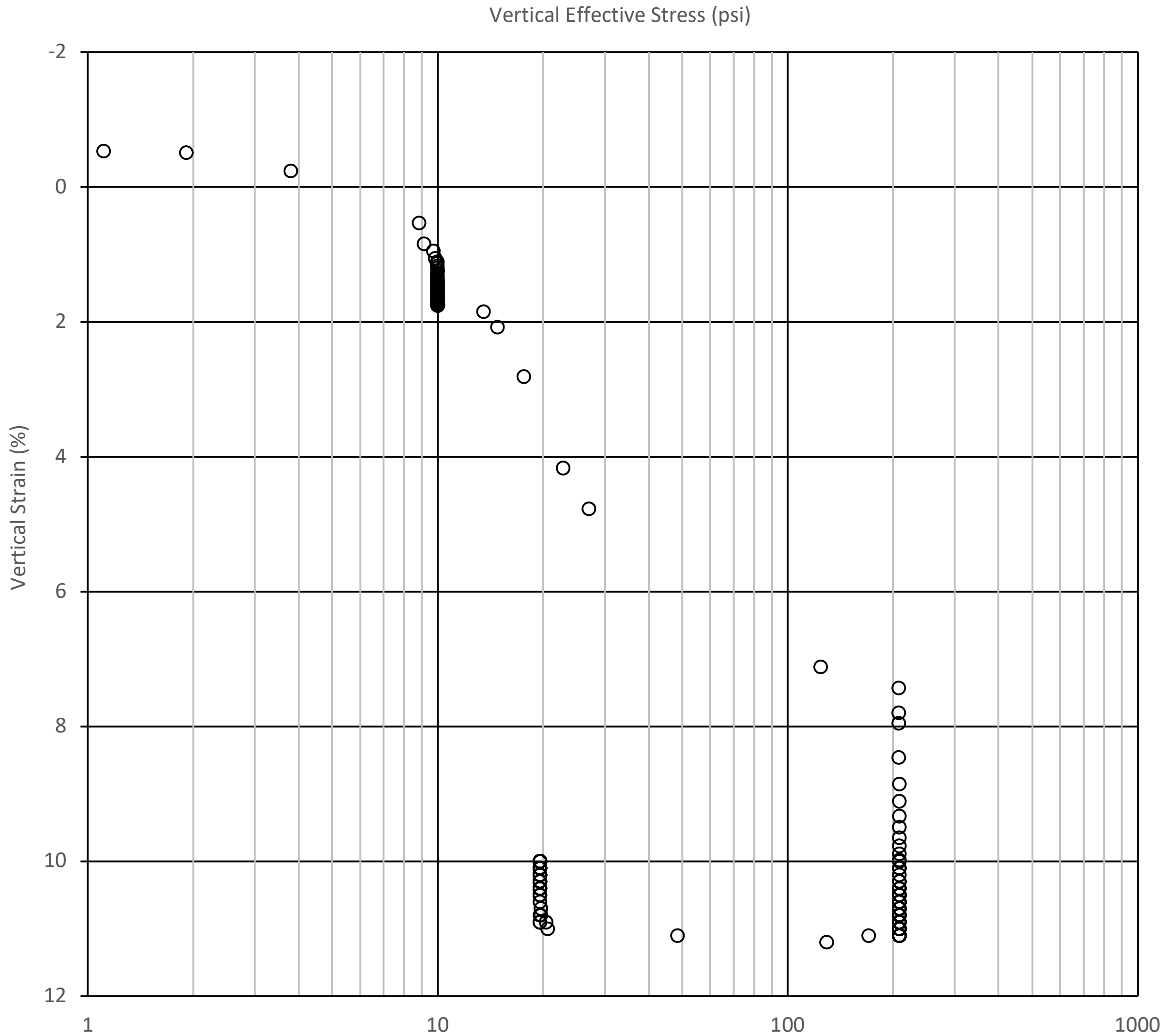


Depth	W.C. (%)		Atterberg Limits				
(ft)	Before	After	LL	PL	PI	Description	USCS
87	77	72	31	15	16	LEAN CLAY	CL

Partical-size Distribution		
% Gravel	% Sand	% Fines
NT	NT	NT

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.25
Total Unit Weight (pcf)	93.57
Degree of Saturation (%)	98.68
Void Ratio (e_0)	1.970

Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

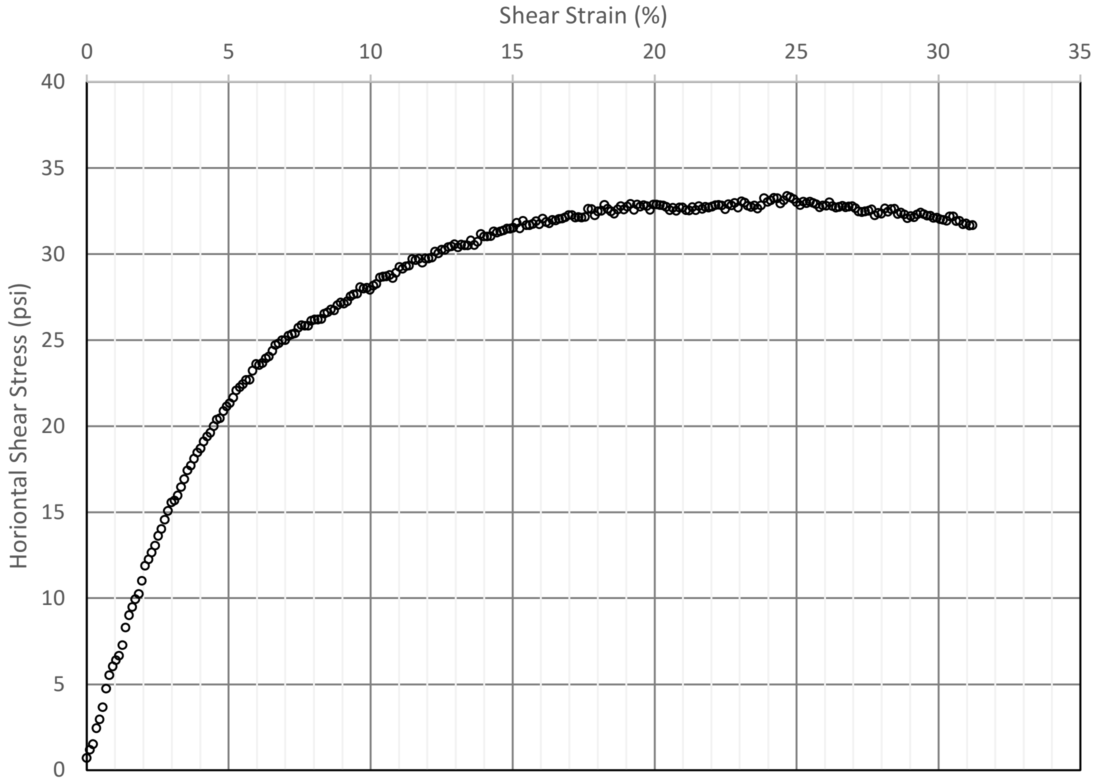
Squalicum FP
Bremerton, WA

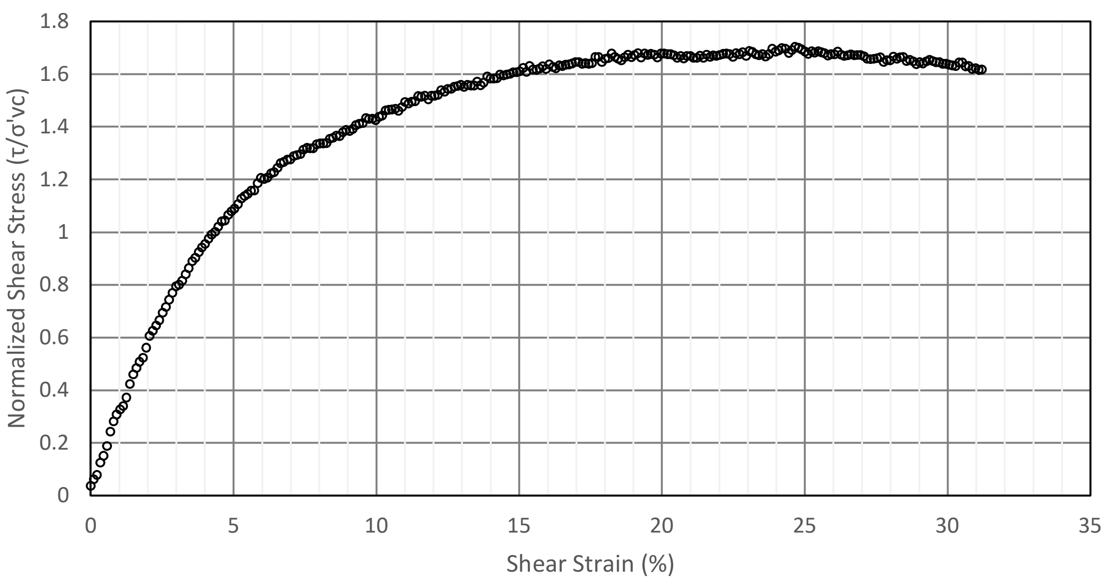
Axial Strain Versus Logarithm of Vertical Effective Stress for H-4si-21 PS-6 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-6-1





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

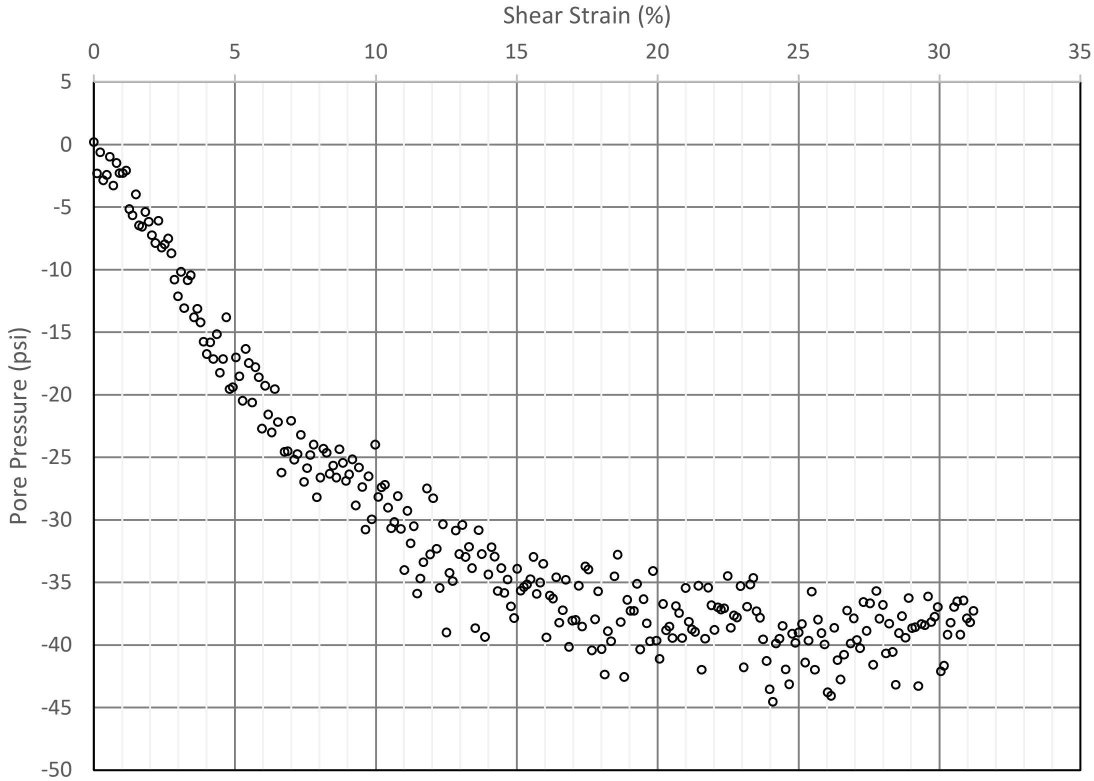
Squalicum FP
Bremerton, WA

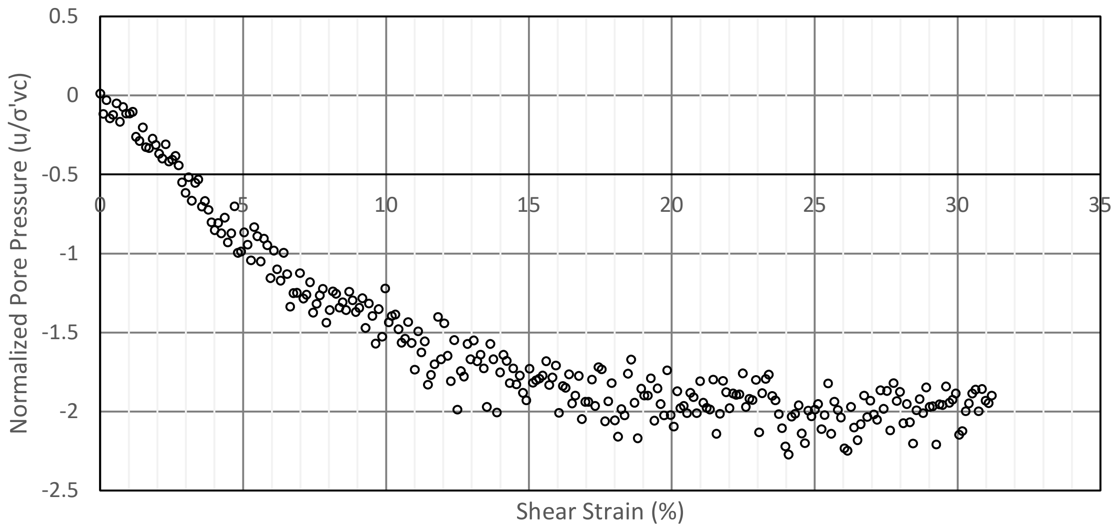
Horizontal Shear Stress and Normalized Shear Stress for H-4si-21 PS-6 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-6-2





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

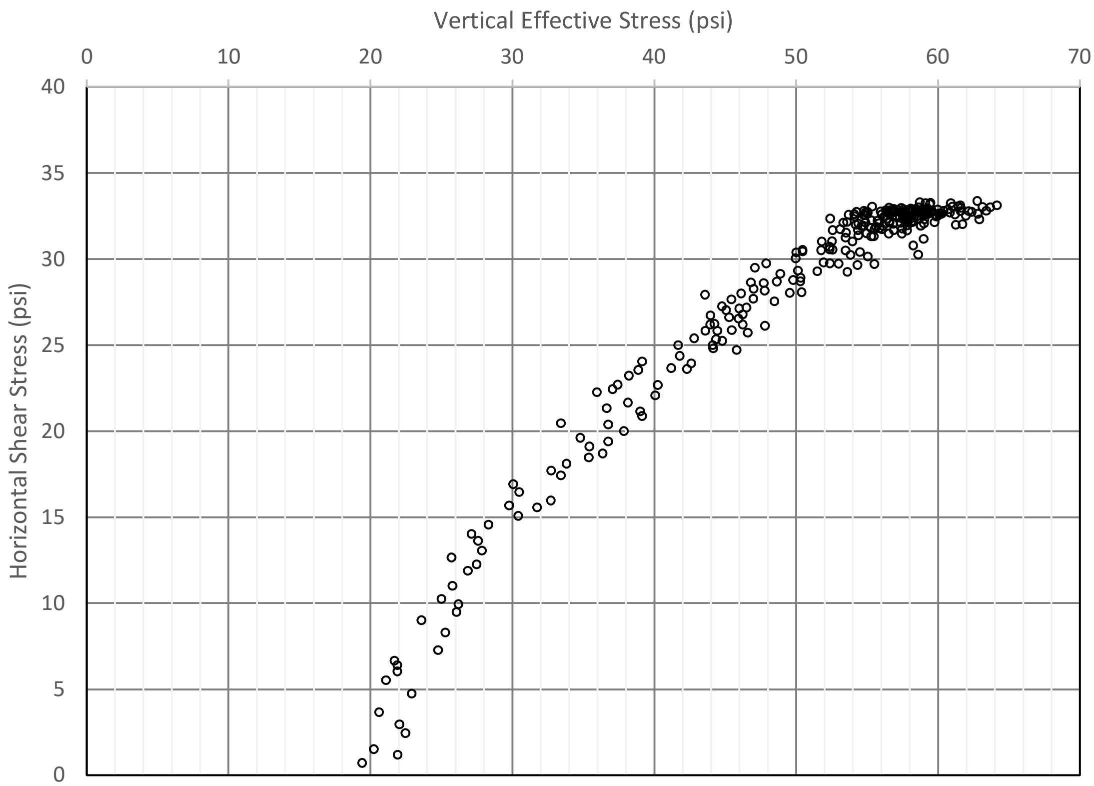
Squalicum FP
Bremerton, WA

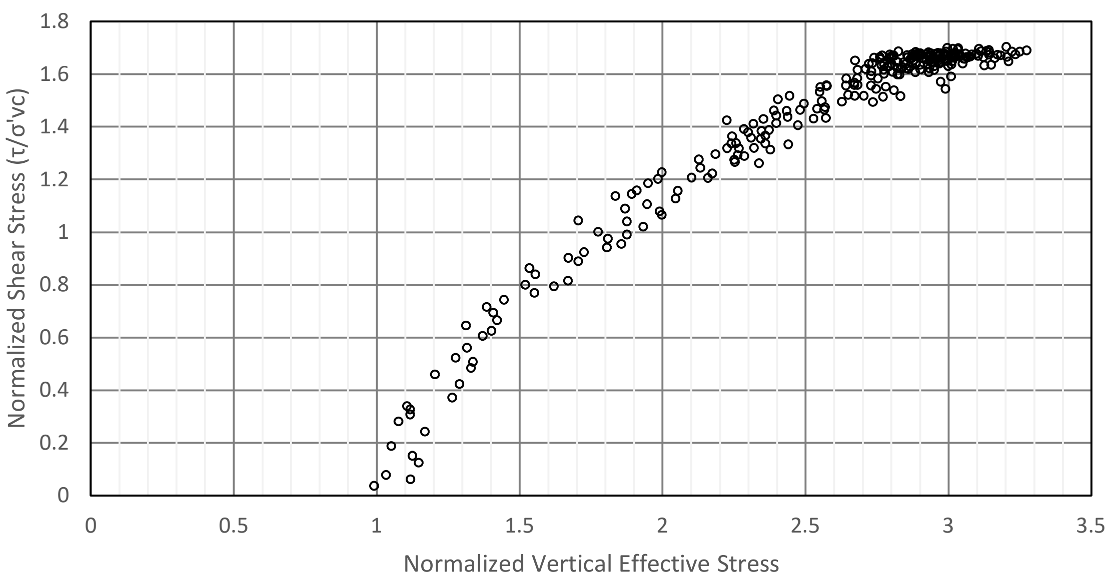
Pore Pressure and Normalized Pore Pressure Versus Shear Strain for H-4si-21 PS-6 Specimen#1 DSS

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-6-3





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bremerton, WA

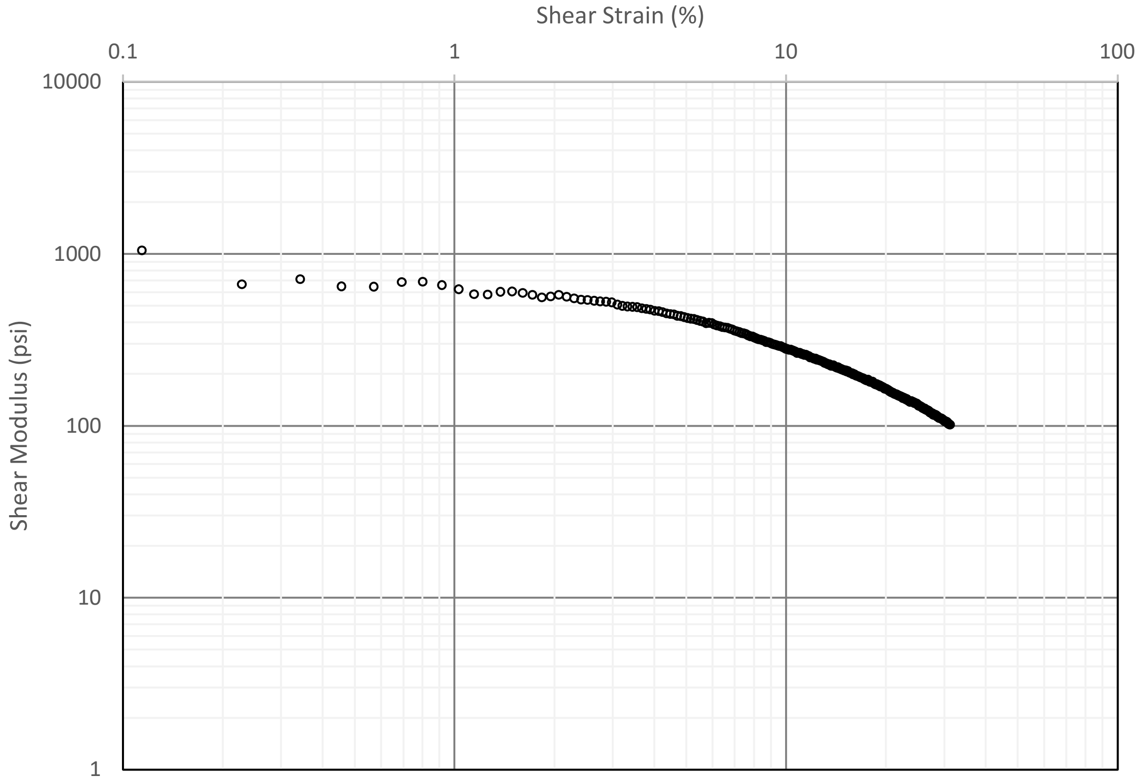
Horizontal and Normalized Shear Stress Versus Vertical and Normalized Vertical Effective Stress for H-4si-21 PS-6 Specimen#1 DSS

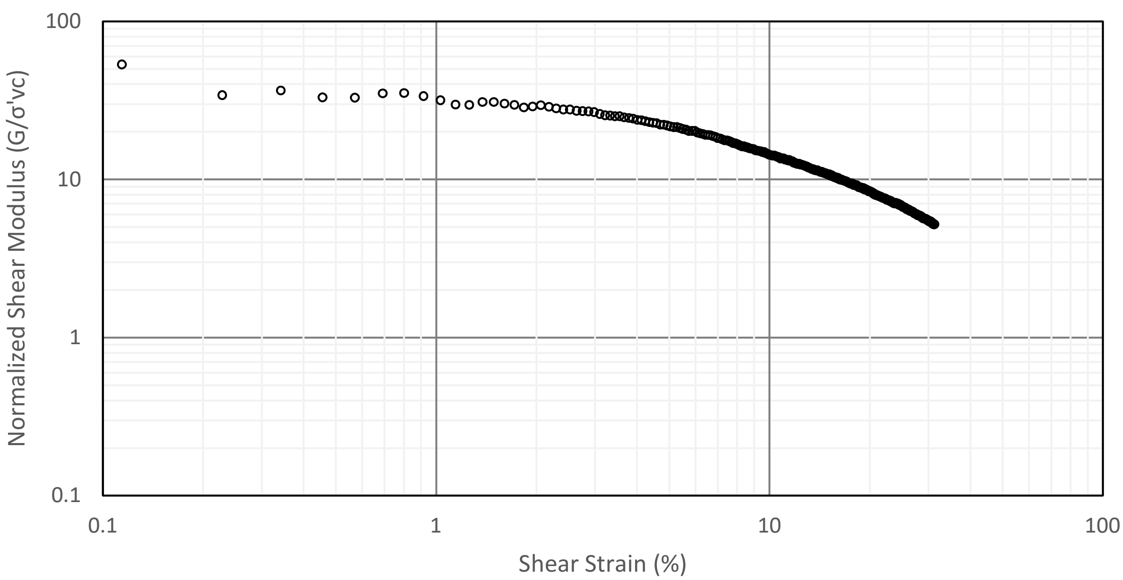
Job Number: 19501-27

06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-6-4





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bremerton, WA

Shear Modulus and Normalized Shear Modulus Versus Shear Strain for H-4si-21 PS-6 Specimen#1 DSS

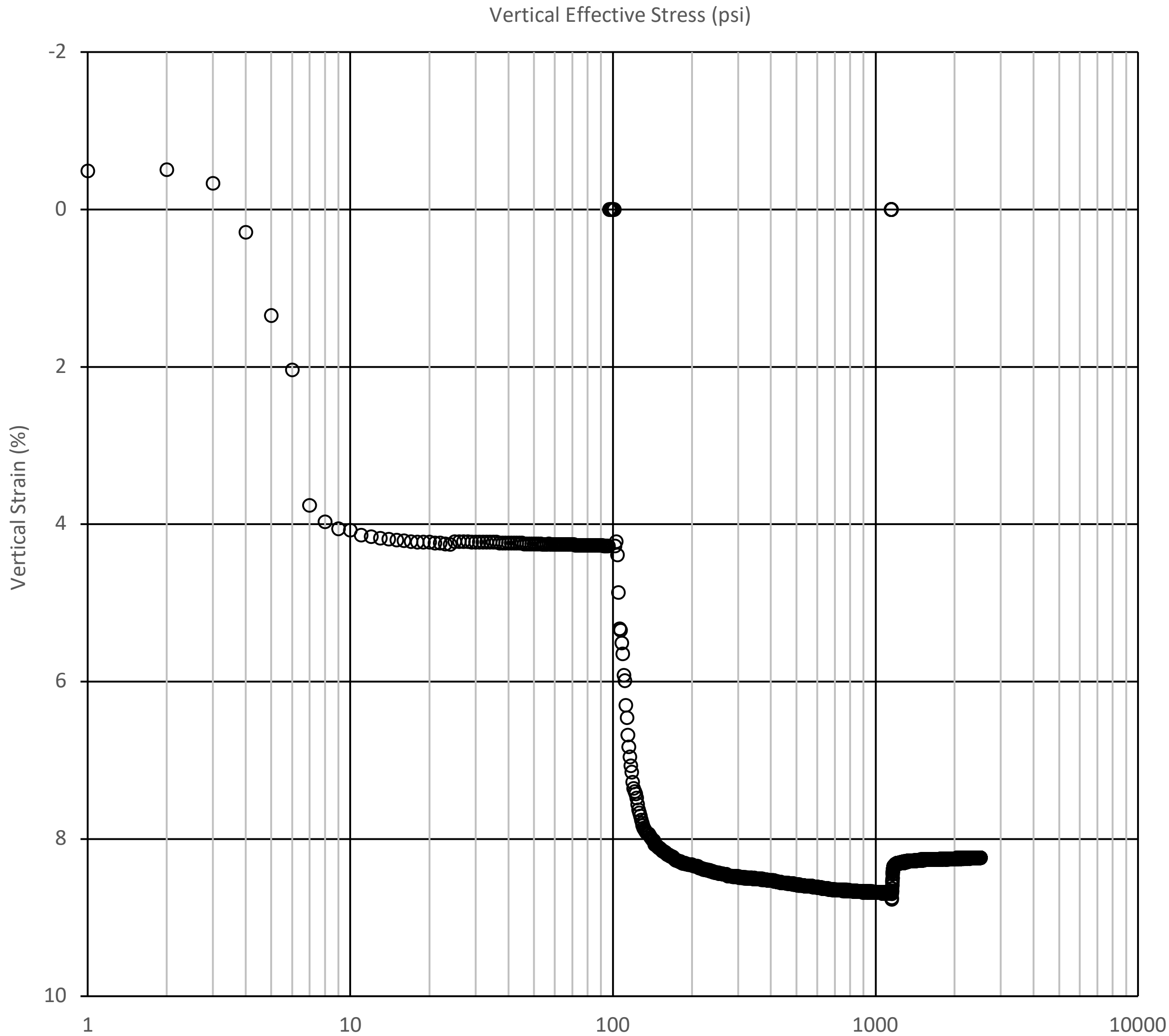
Job Number: 19501-27

06/21

HARTCROWSER
A division of Haley & Aldrich

Figure

C-6-5



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
30.5	77	72	28	14	14	LEAN CLAY	CL

Partical-size Distribution		
% Gravel	% Sand	% Fines
NT	NT	NT

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.25
Total Unit Weight (pcf)	93.57
Degree of Saturation (%)	98.68
Void Ratio (e_0)	1.970

Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

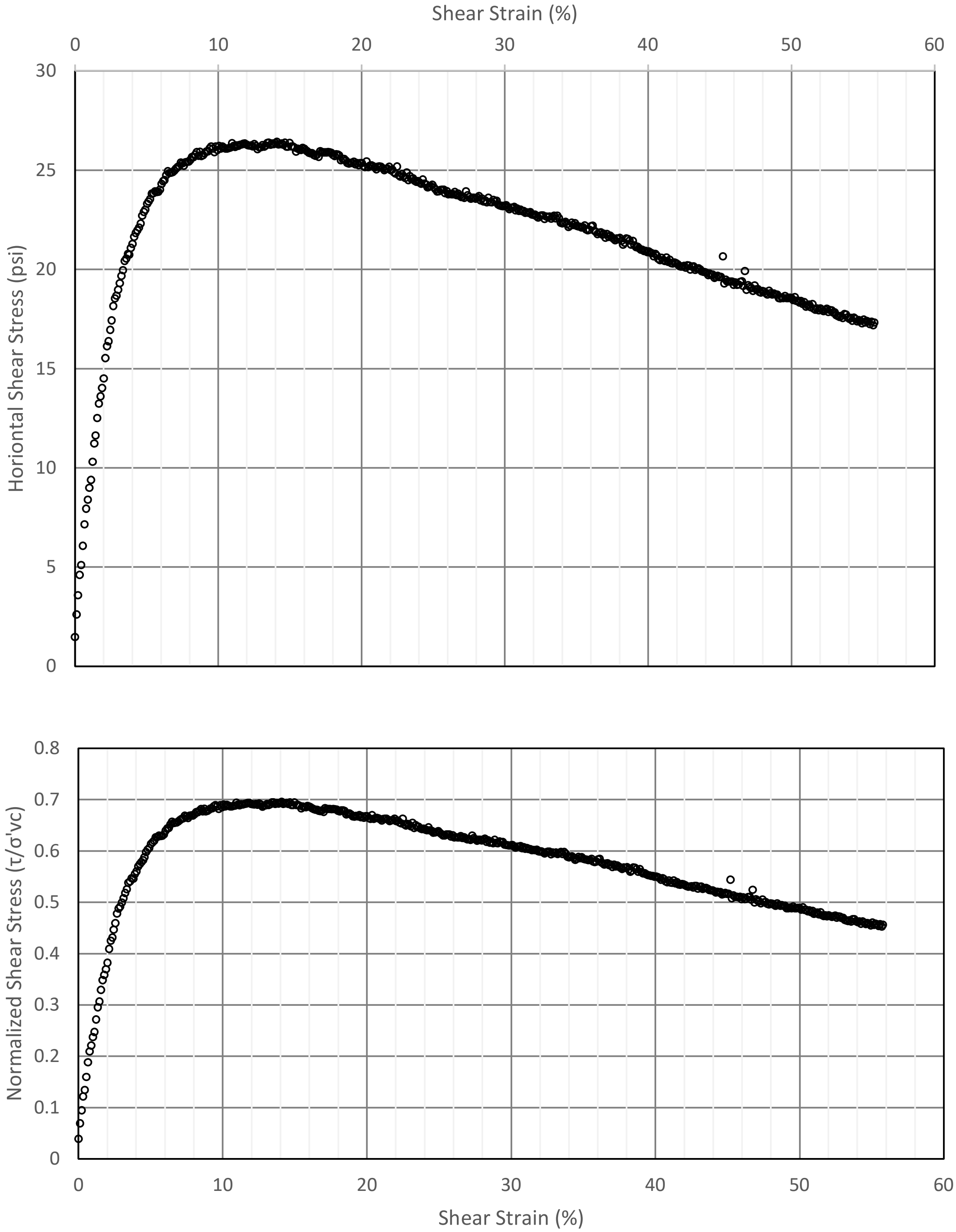
Squalicum FP
Bellingham, WA

Axial Strain Versus Logarithm of Vertical Effective Stress for H-4si-21 PS-18 Specimen#1 DSS

Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-7-1



Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

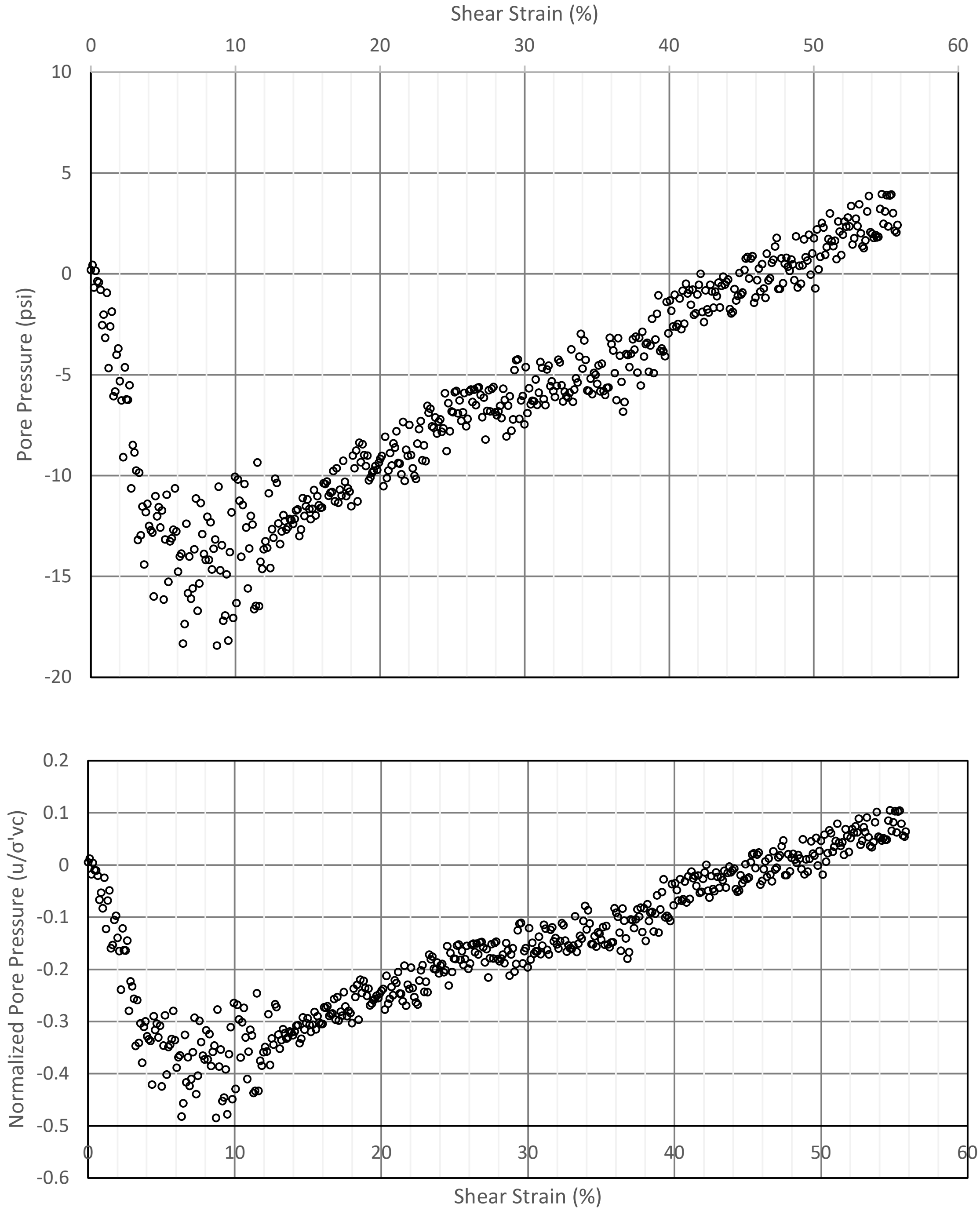
Squalicum FP
Bellingham, WA

Horizontal Shear Stress and Normalized Shear Stress for H-4si-21 PS-18 Specimen#1 DSS

Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-7-2



Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bellingham, WA

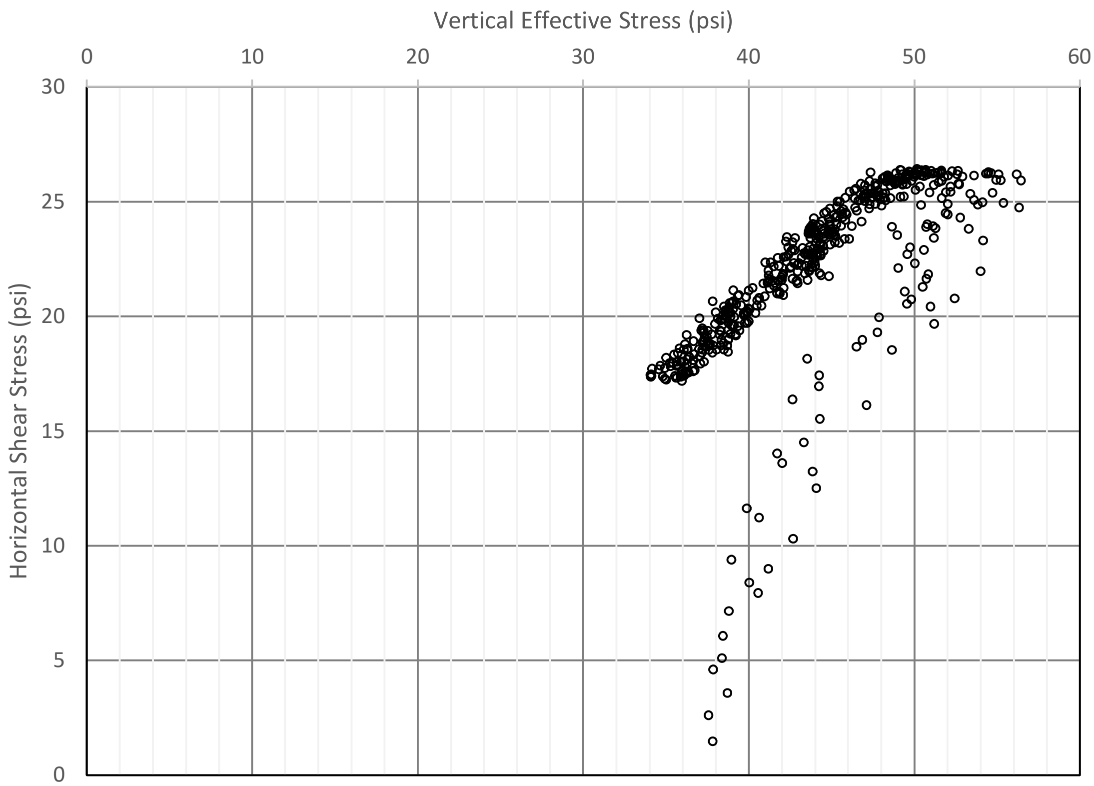
Pore Pressure and Normalized Pore Pressure Versus Shear Strain for H-4si-21 PS-18 Specimen#1 DSS

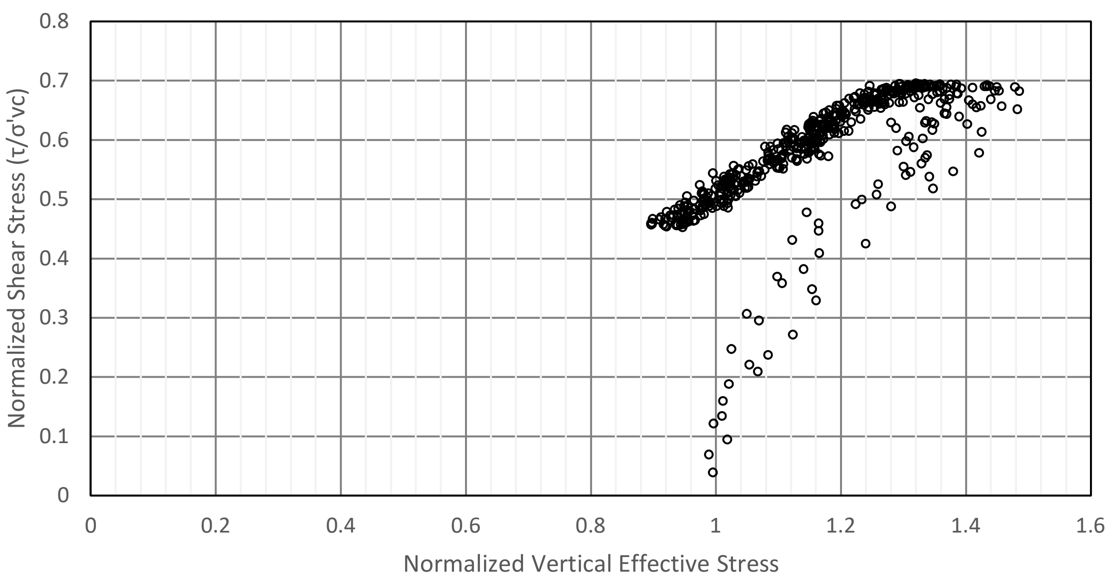
Job Number: 19501-27

10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-7-3





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bellingham, WA

Horizontal and Normalized Shear Stress Versus Vertical and Normalized Vertical Effective Stress for H-4si-21 PS-18 Specimen#1 DSS

Job Number: 19501-27

10/21

HARTCROWSER
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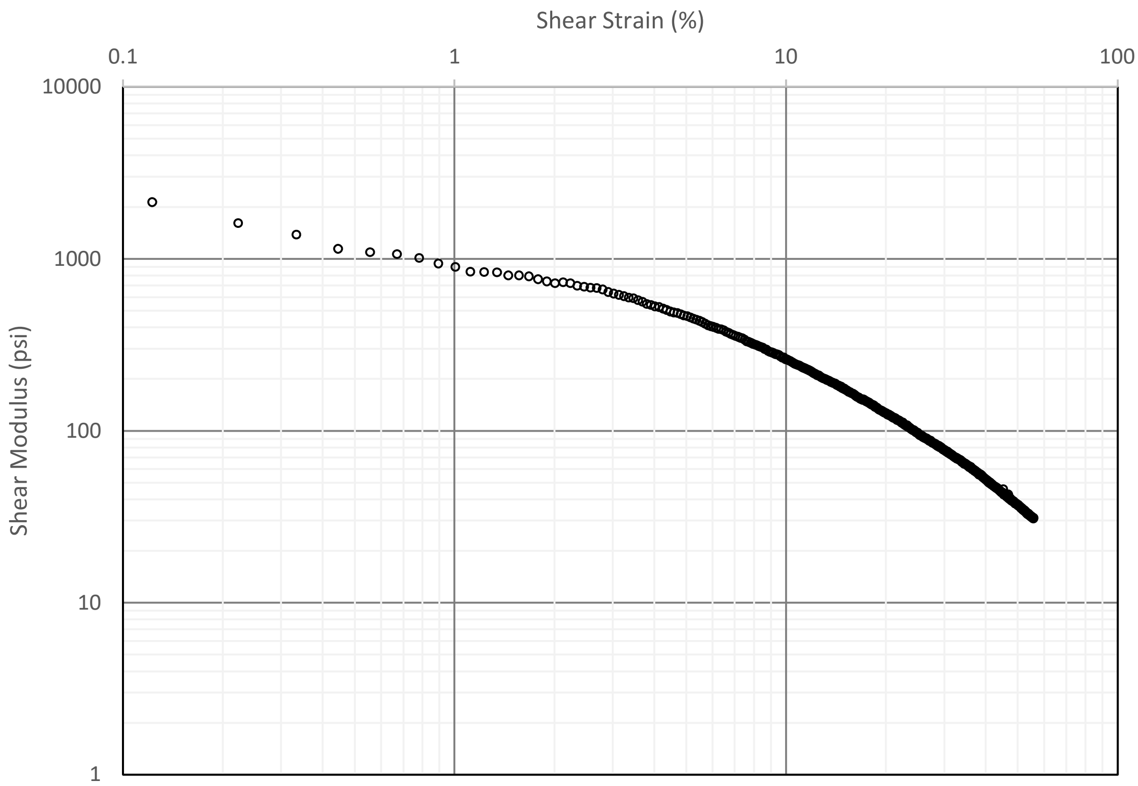
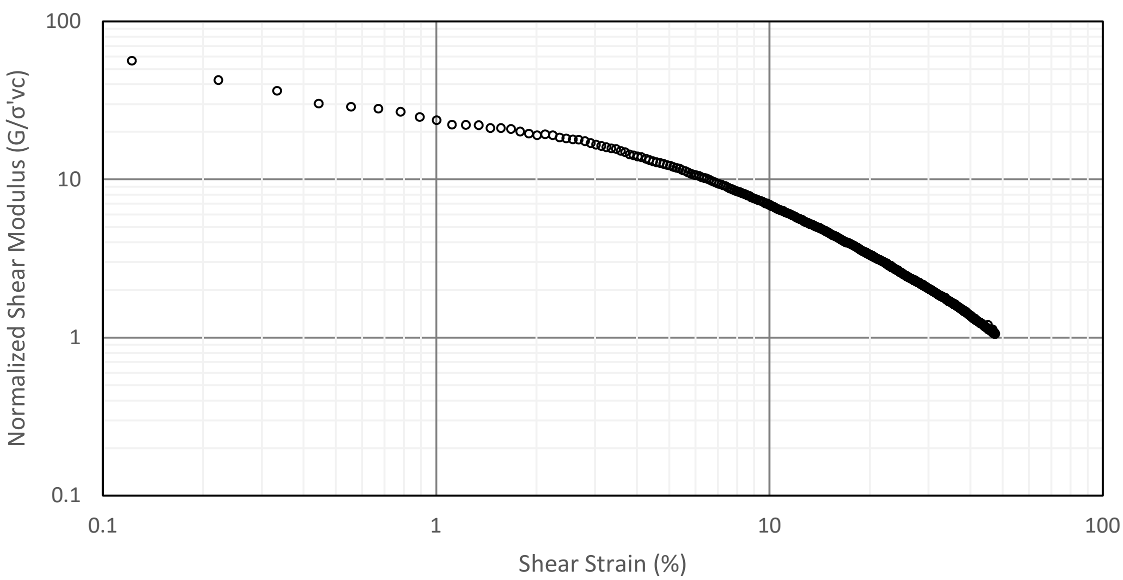
Figure
C-7-4

ML 07/18/2022 \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\Deliverables In-Basket\FINAL Report\Attachments\Appendices\Appendix B & C - Laboratory Data\Appendix C\DSS\DSS_I





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bellingham, WA

Shear Modulus and Normalized Shear Modulus Versus Shear Strain for H-4si-21 PS-18 Specimen#1 DSS

Job Number: 19501-27 10/21

HARTCROWSER
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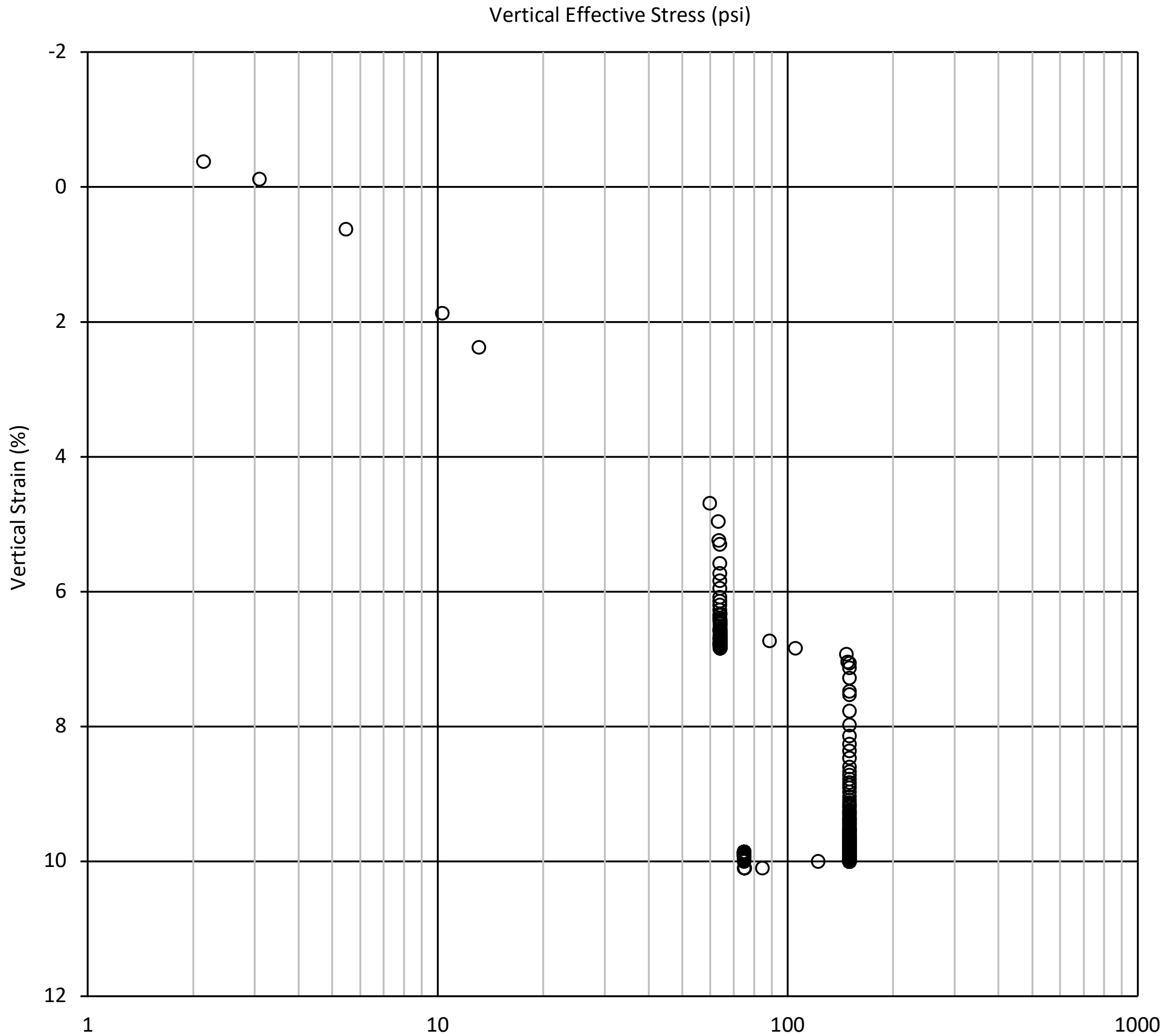
Figure
C-7-5

ML 07/18/2022 \\haleyaldrich.com\share\sea_projects\Notebooks\1950127_SR 542_Squalicum_Creek_FP_XL6093\Deliverables In-Basket\FINAL Report\Attachments\Appendices\Appendix B & C - Laboratory Data\Appendix C\DSS\DS:



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
86.7	114	96	38	17	21	LEAN CLAY with SAND	CL

Partical-size Distribution		
% Gravel	% Sand	% Fines
NT	NT	76.3

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	3.87
Total Unit Weight (pcf)	85.25
Degree of Saturation (%)	97.58
Void Ratio (e_0)	2.929

Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

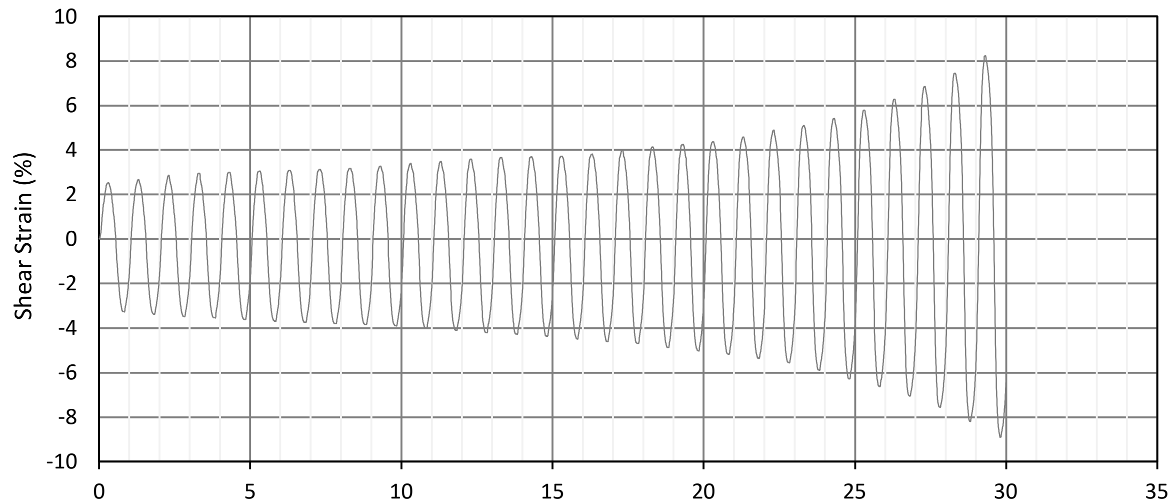
Squalicum FP
Bremerton, WA

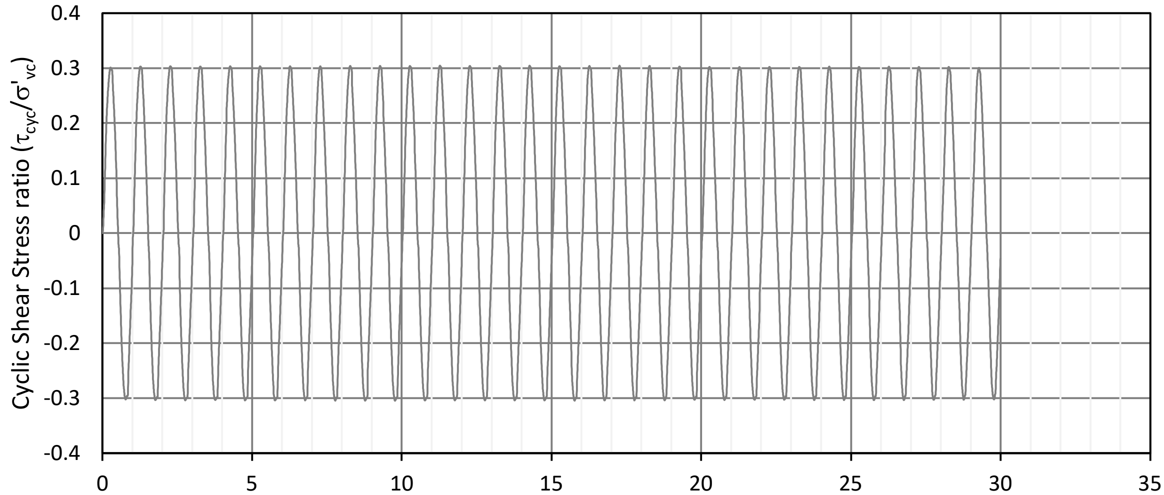
Axial Strain Versus Logarithm of Vertical Effective Stress for H-2p-20 PS-29 stress-controlled CDSS Consolidation Phase

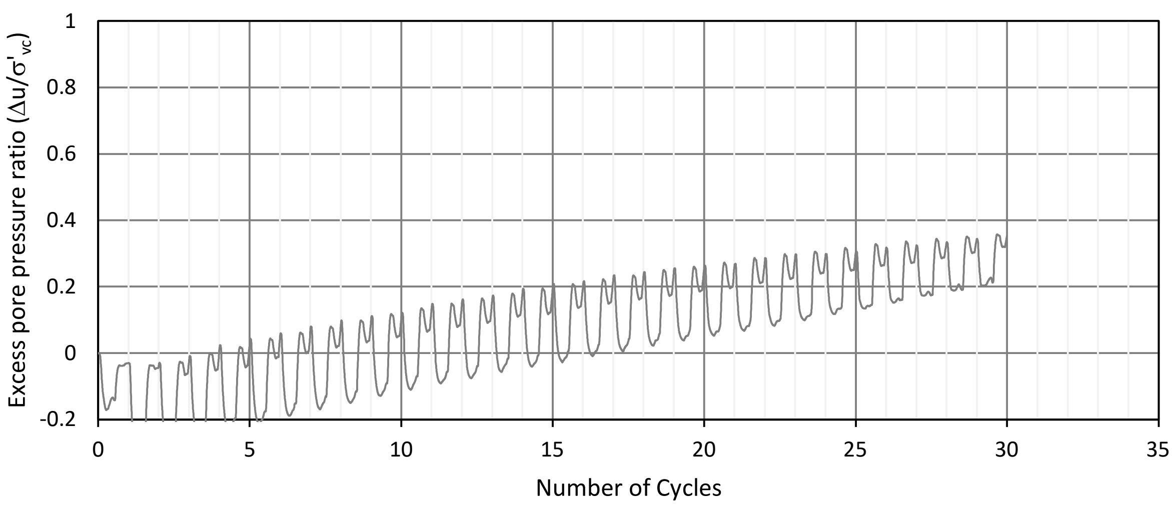
Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-8-1







Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

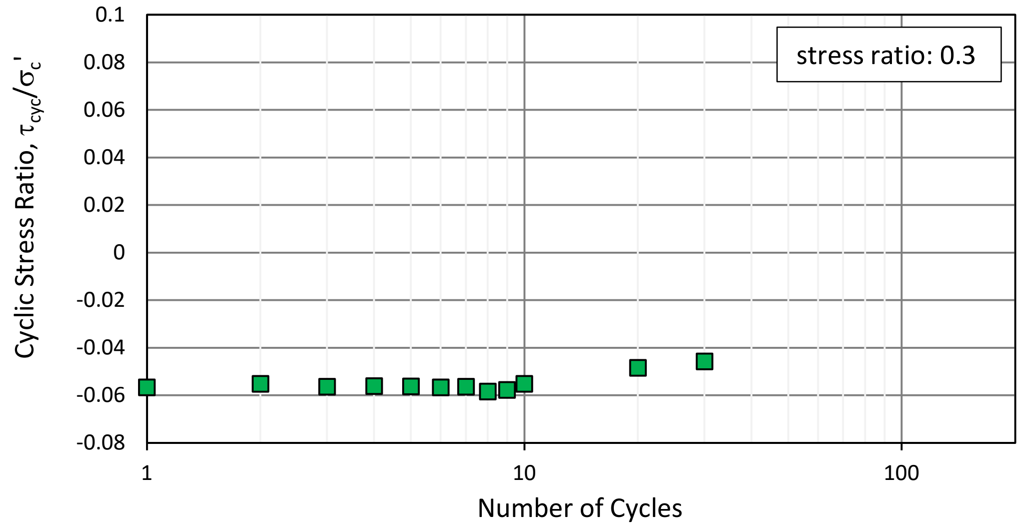
Squalicum FP
Bremerton, WA

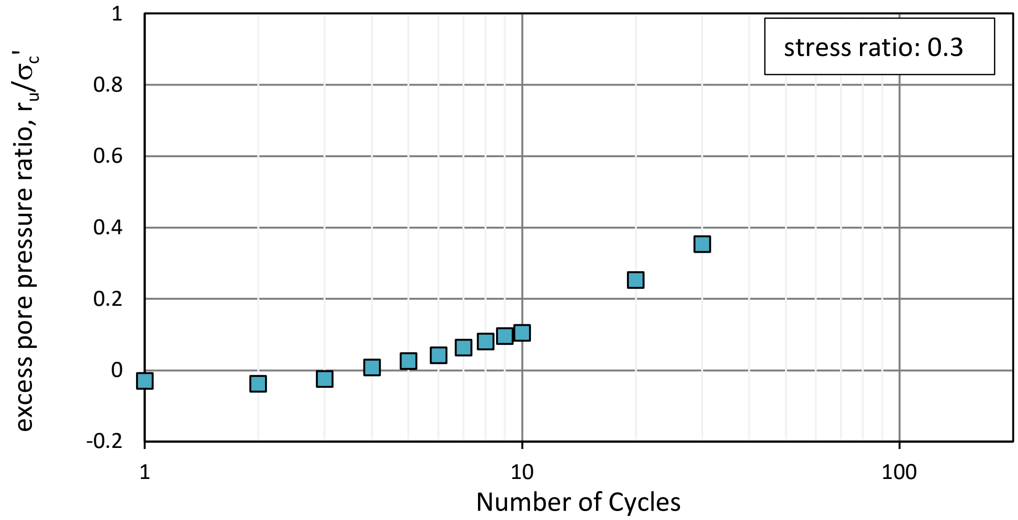
Cyclic shear phase data for H-2p-20 PS-29 stress-controlled CDSS Cyclic Phase

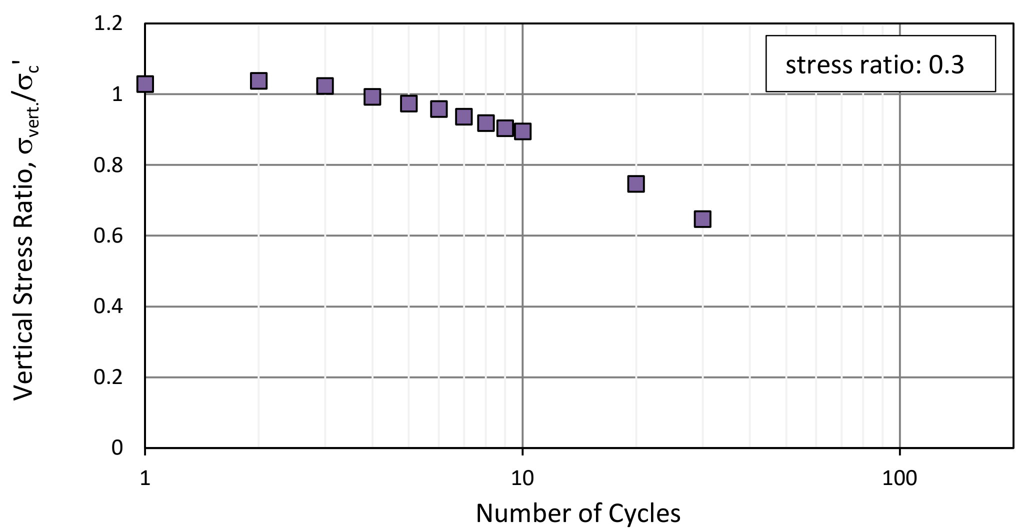
Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-8-2a







Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

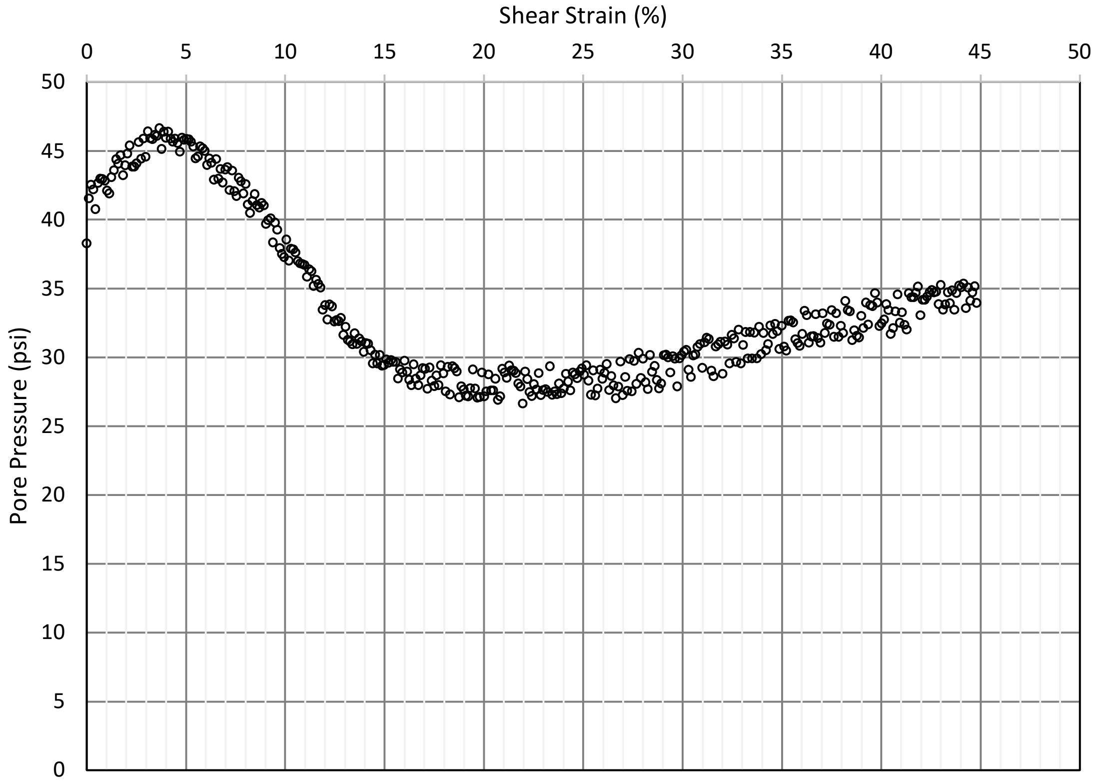
Squalicum FP
Bremerton, WA

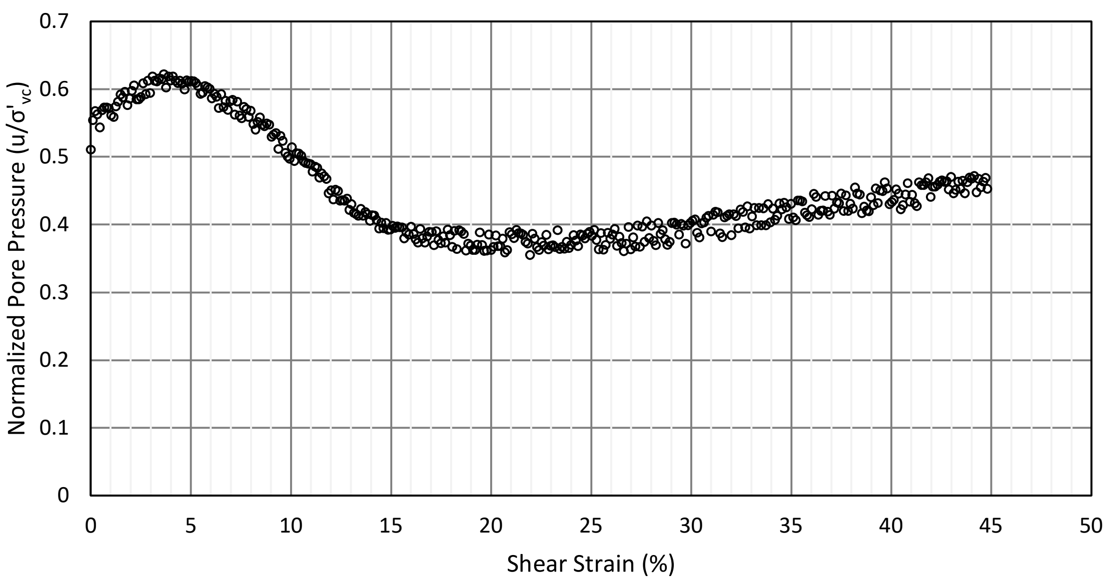
Cyclic Loop for HC20-DD7-05 PS-29 stress-controlled CDSS Cyclic Phase

Job Number: 19526-01 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-8-2b





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

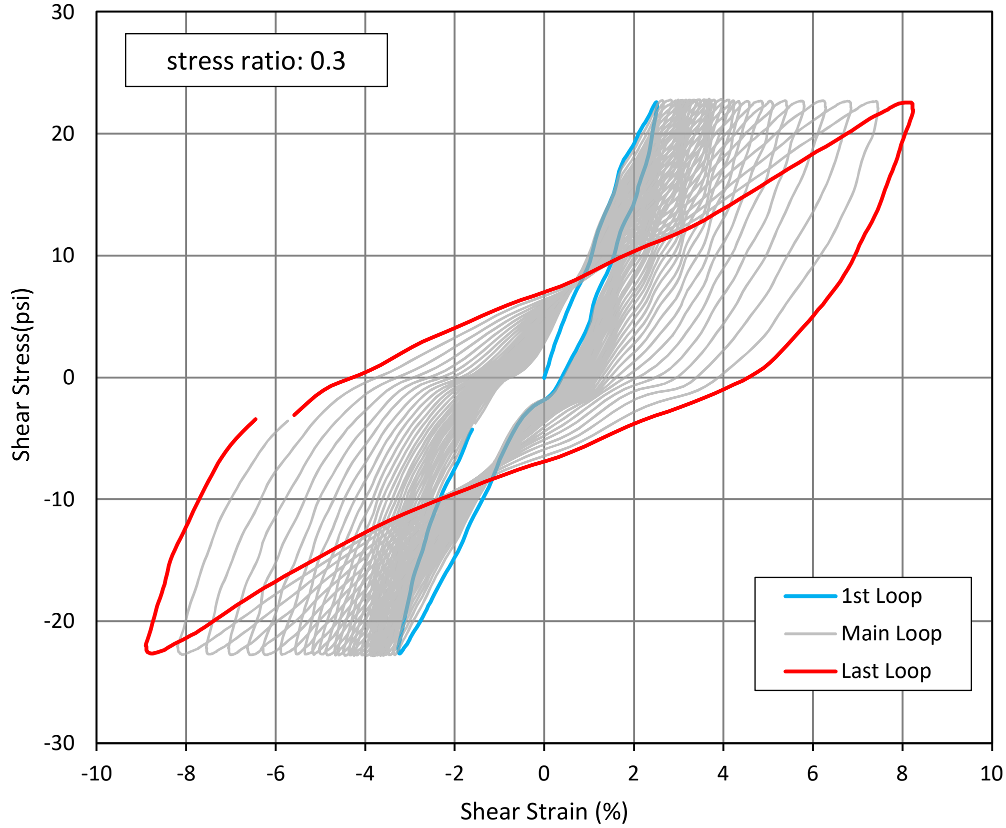
Squalicum FP
Bremerton, WA

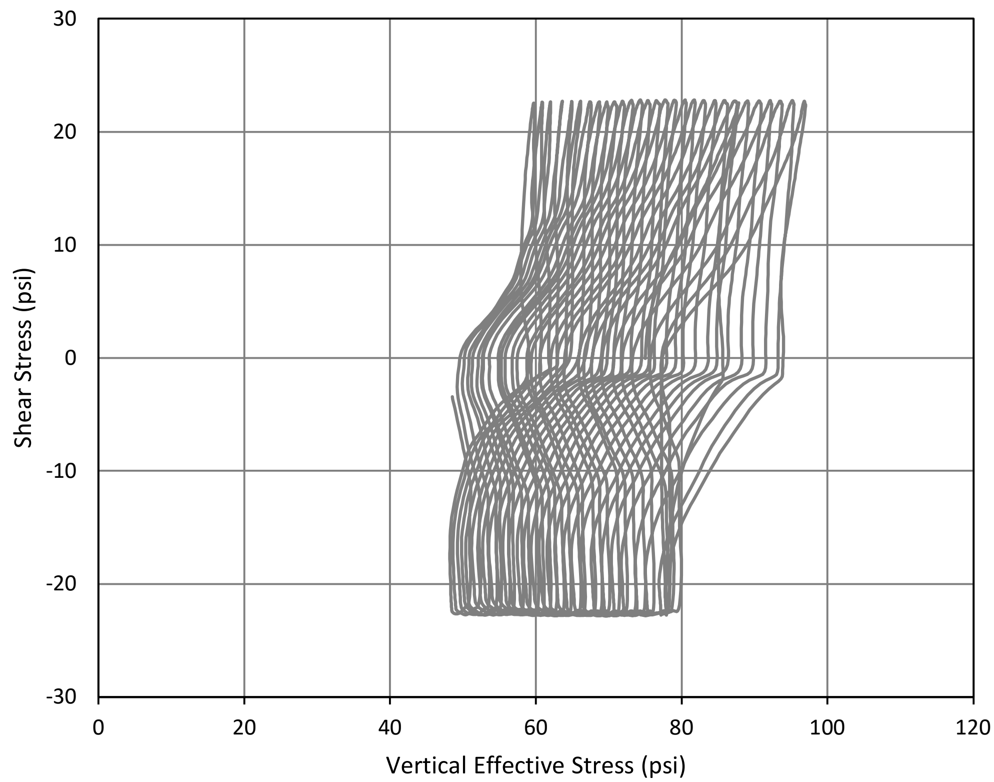
Pore Pressure and Normalized Pore Pressure Versus Shear Strain for H-2p-20 PS-29 stress-controlled CDSS Post-Cyclic Shear Phase

Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-8-5





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Bremerton, WA

Cyclic Loop for H-2p-20 PS-29 stress-controlled CDSS Cyclic Phase

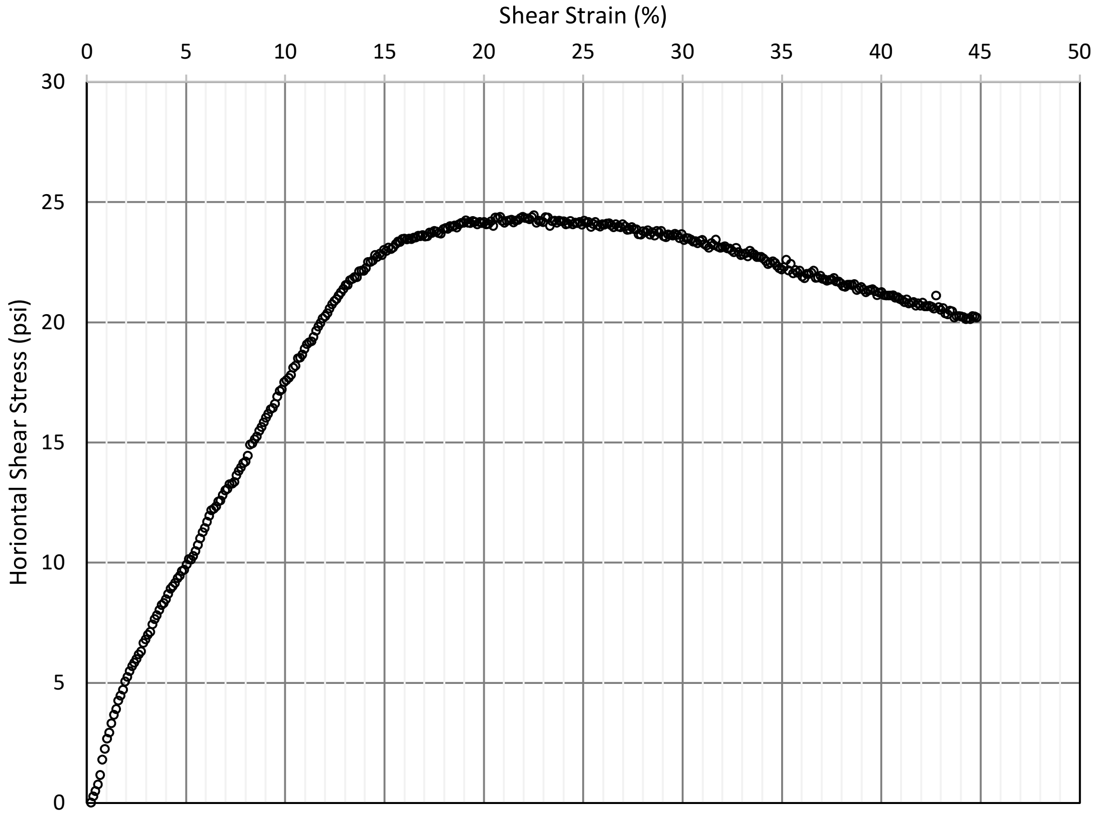
Job Number: 19526-01

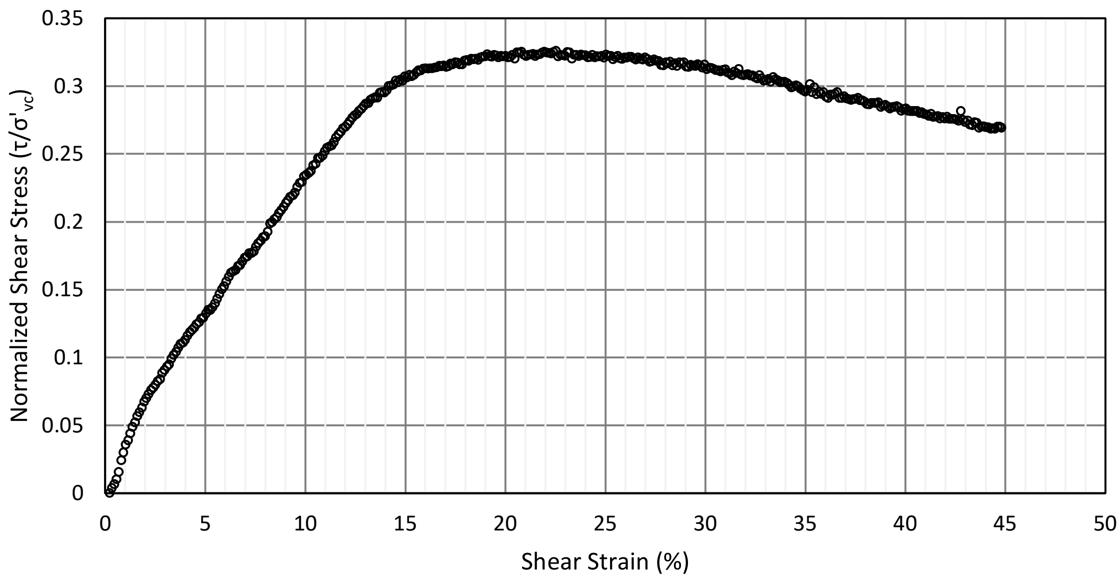
10/21

HARTCROWSER
A division of Haley & Aldrich

Figure

C-8-3





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section. Post-cyclic direct simple shear test stress and strain are measured relative to the state of stress of the soil specimen at the end of the cyclic phase

σ'vc = Vertical effective stress at the end of consolidation

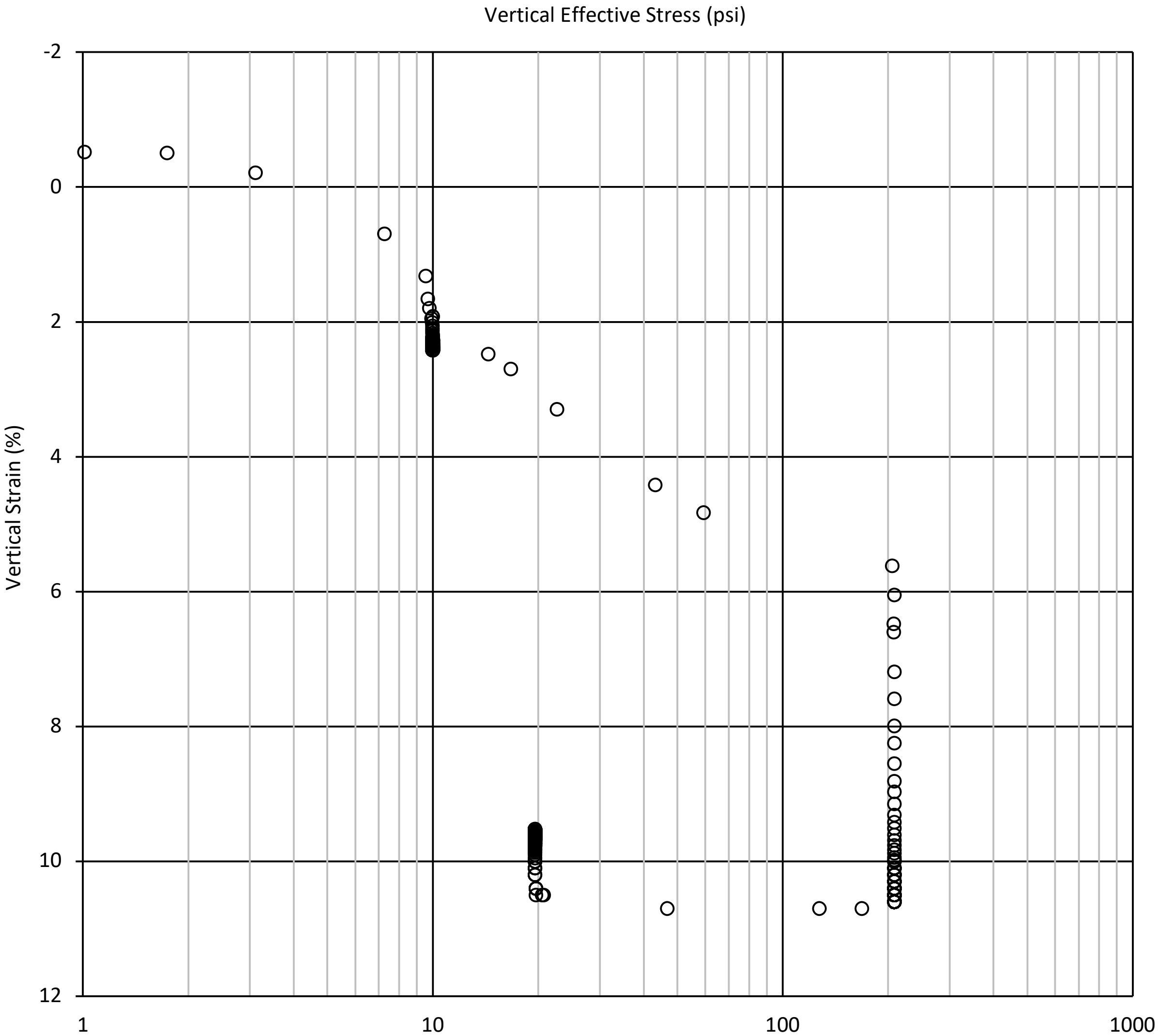
Squalicum FP
Bremerton, WA

Horizontal Shear Stress and Normalized Shear Stress for H-2p-20 PS-29 stress-controlled CDSS Post-Cyclic Shear Phase

Job Number: 19501-27 10/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-8-4



Depth	W.C. (%)		Atterberg Limits				
(ft)	Before	After	LL	PL	PI	Description	USCS
0	114	96	31	15	16	Fat Clay	CL

Partical-size Distribution		
% Gravel	% Sand	% Fines
NT	NT	NT

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	3.87
Total Unit Weight (pcf)	85.25
Degree of Saturation (%)	97.58
Void Ratio (e_0)	2.929

Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

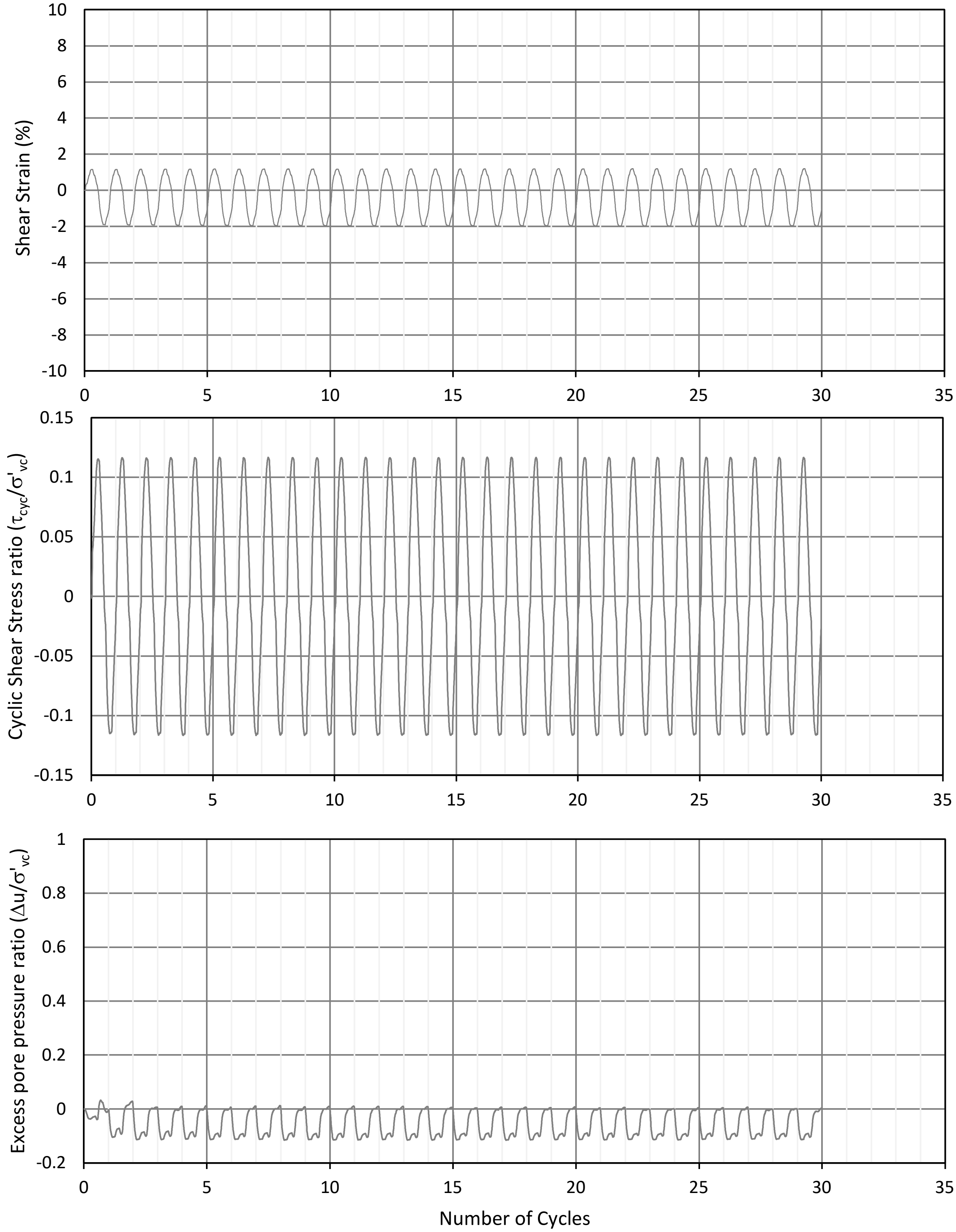
Squalicum FP
Squalicum , WA

Axial Strain Versus Logarithm of Vertical Effective Stress for H-4si-21 PS-6 stress-controlled CDSS Consolidation Phase

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-1



Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Squalicum , WA

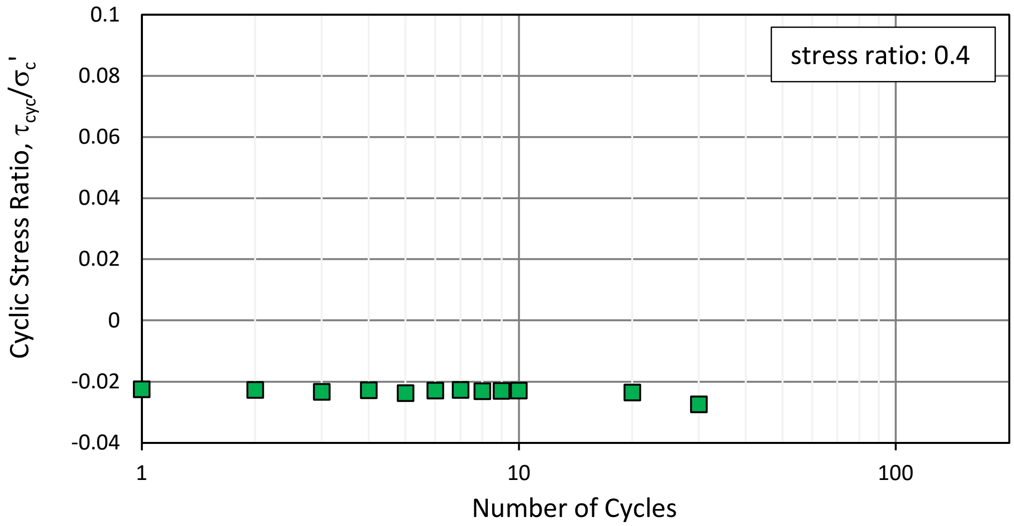
Cyclic shear phase data for H-4si-21 PS-6 stress-controlled CDSS Cyclic Phase

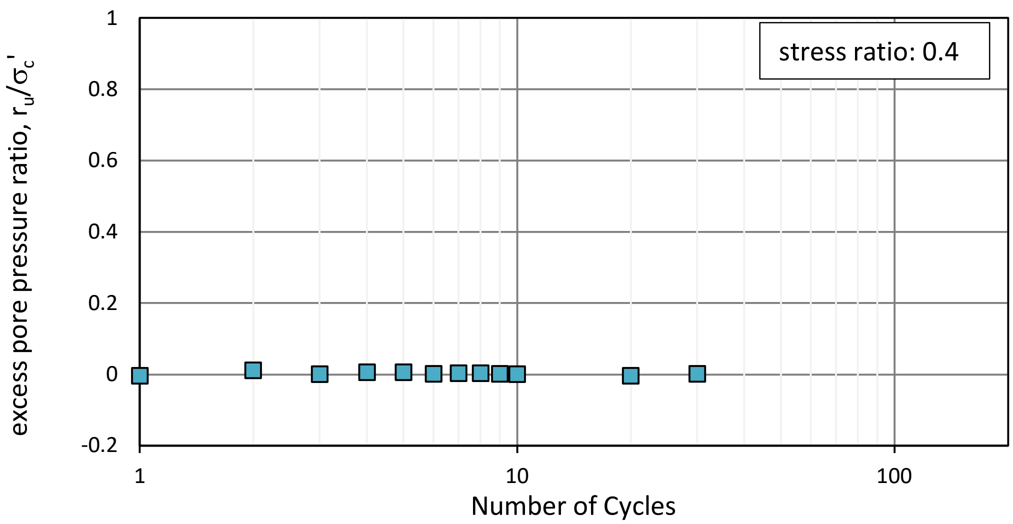
Job Number: 19501-27

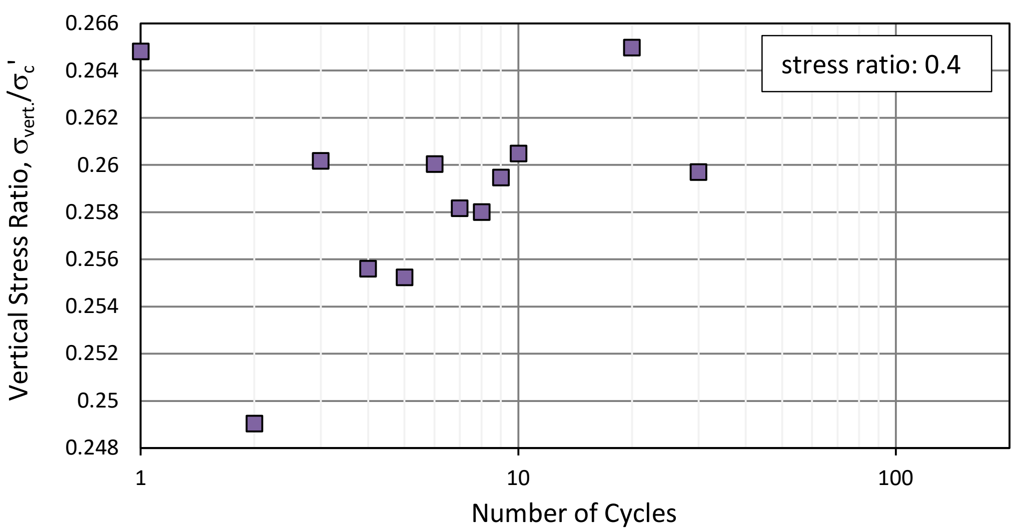
06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-2a







Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Squalicum , WA

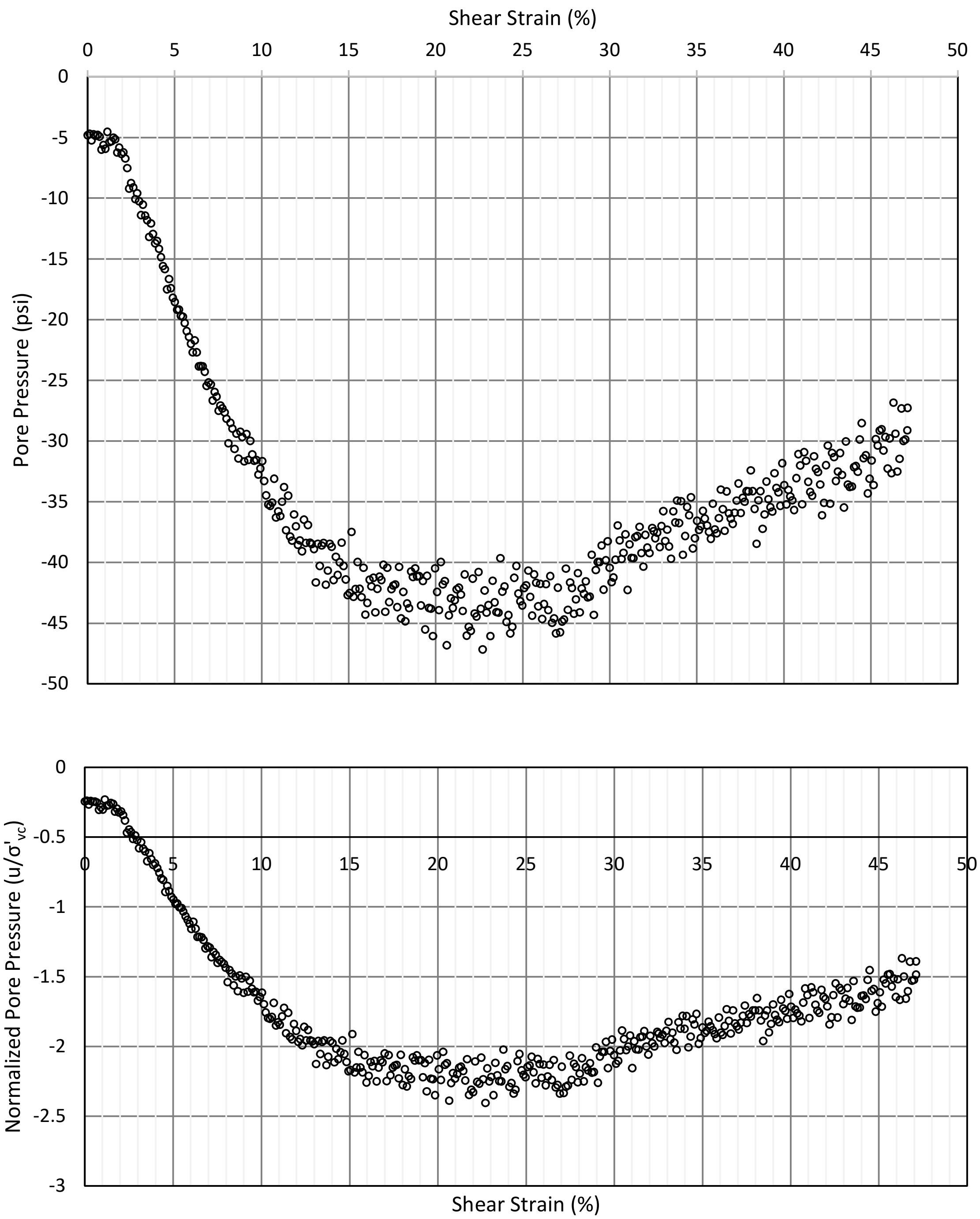
Cyclic Loop for HC20-DD7-05 PS-6 stress-controlled CDSS Cyclic Phase

Job Number: 19526-01

06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-2b



Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

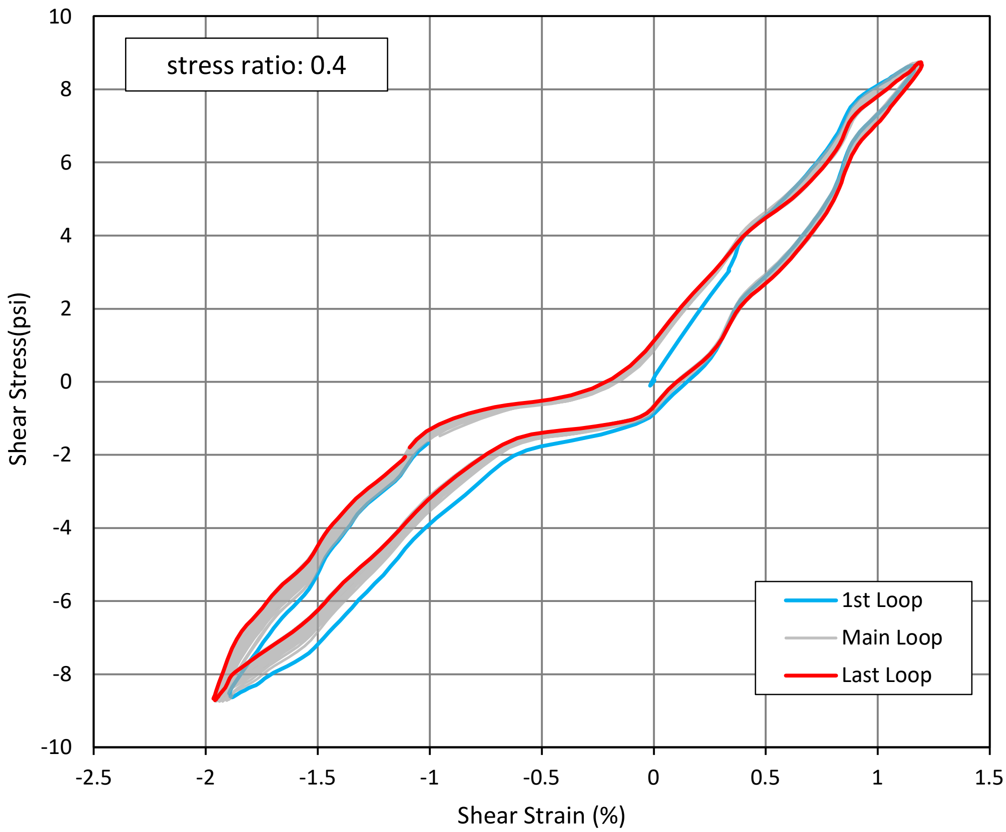
Squalicum FP
Squalicum , WA

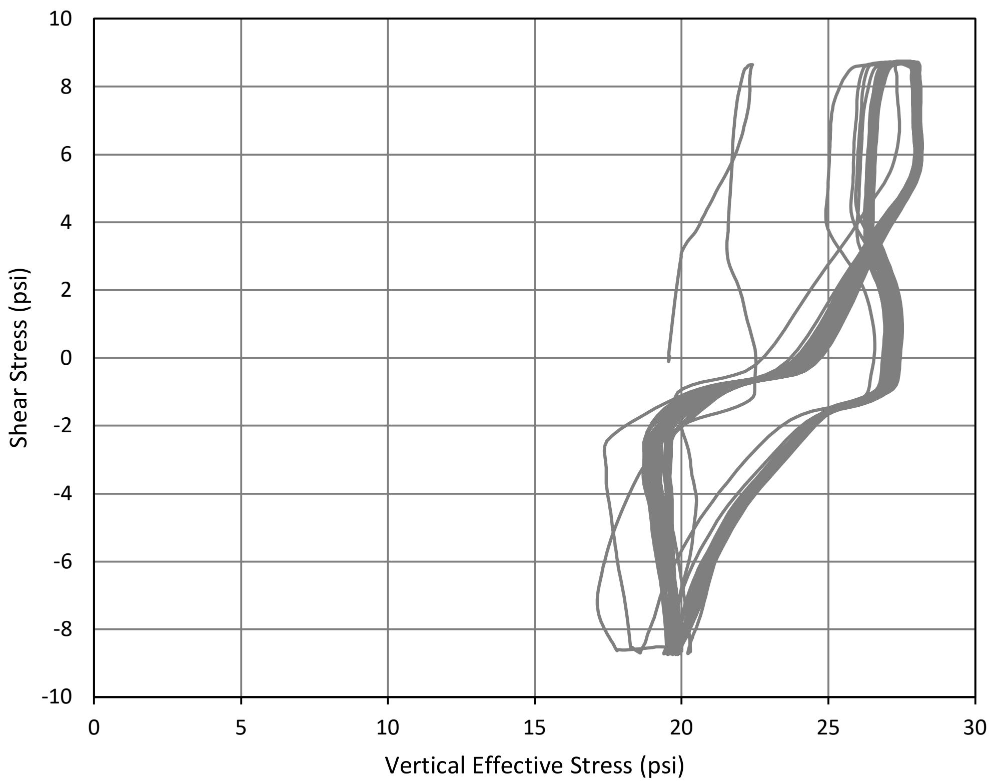
Pore Pressure and Normalized Pore Pressure Versus Shear Strain for H-4si-21 PS-6 stress-controlled CDSS Post-Cyclic Shear Phase

Job Number: 19501-27 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-5





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section.

σ'vc = Vertical effective stress at the end of consolidation

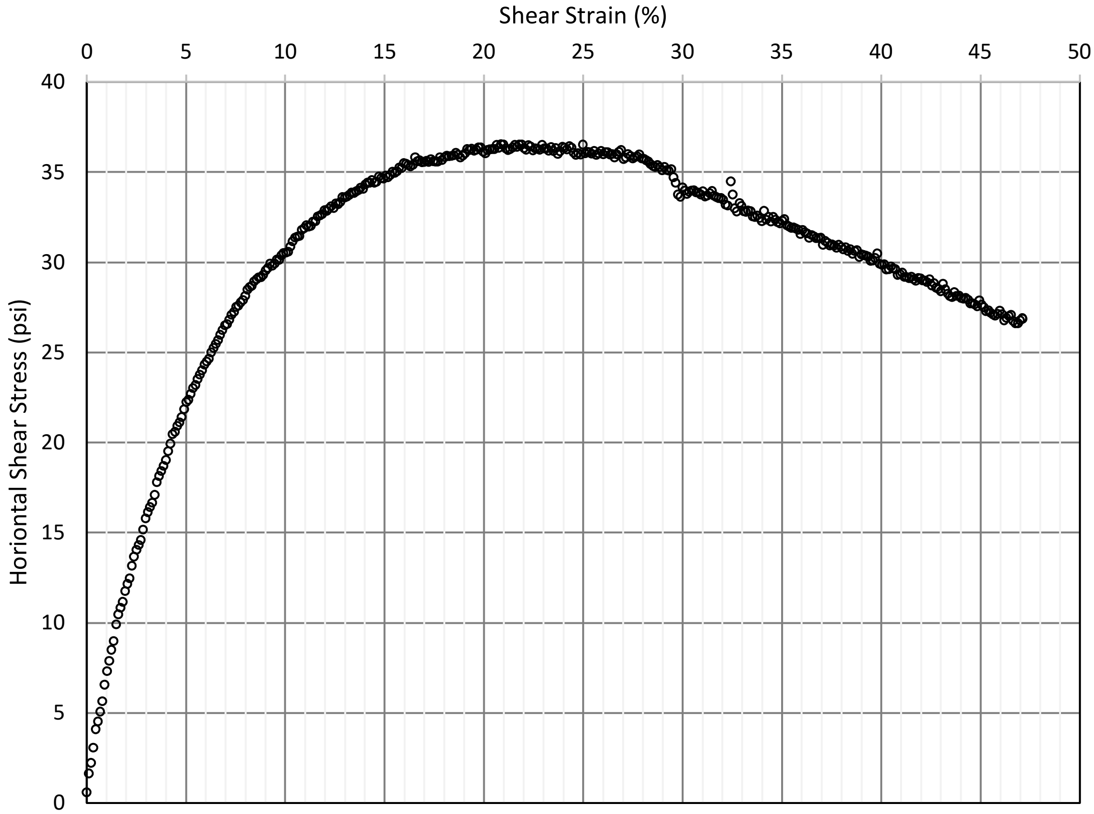
Squalicum FP
Squalicum , WA

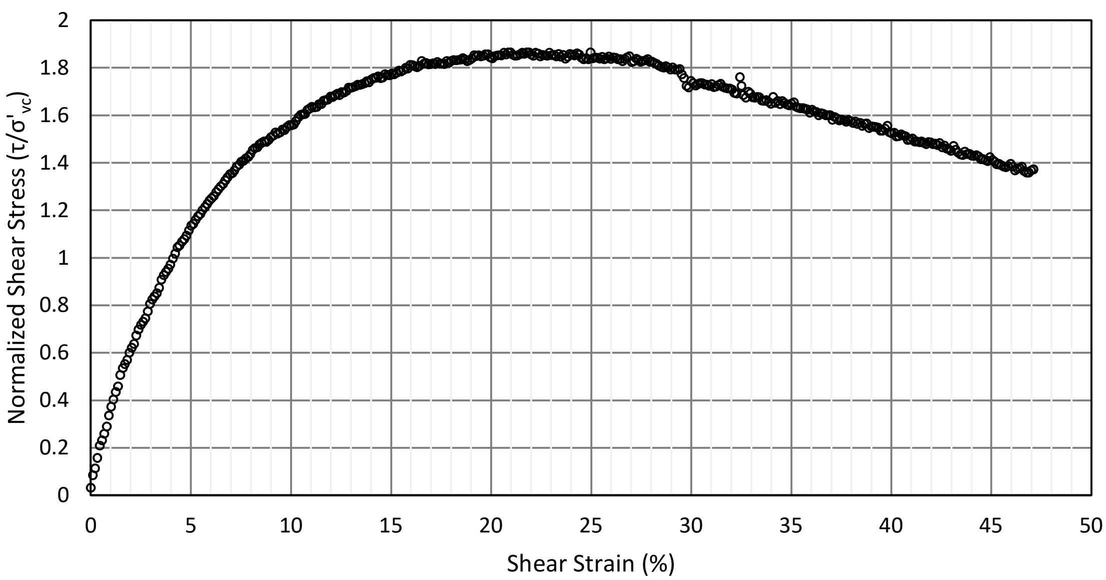
Cyclic Loop for H-4si-21 PS-6 stress-controlled CDSS Cyclic Phase

Job Number: 19526-01 06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-3





Sample preparation and comments: Thin-walled tube specimen cut from section of tube sample; delaminated and pushed to extrude from tube section. Post-cyclic direct simple shear test stress and strain are measured relative to the state of stress of the soil specimen at the end of the cyclic phase

σ'vc = Vertical effective stress at the end of consolidation

Squalicum FP
Squalicum , WA

Horizontal Shear Stress and Normalized Shear Stress for H-4si-21 PS-6 stress-controlled CDSS Post-Cyclic Shear Phase

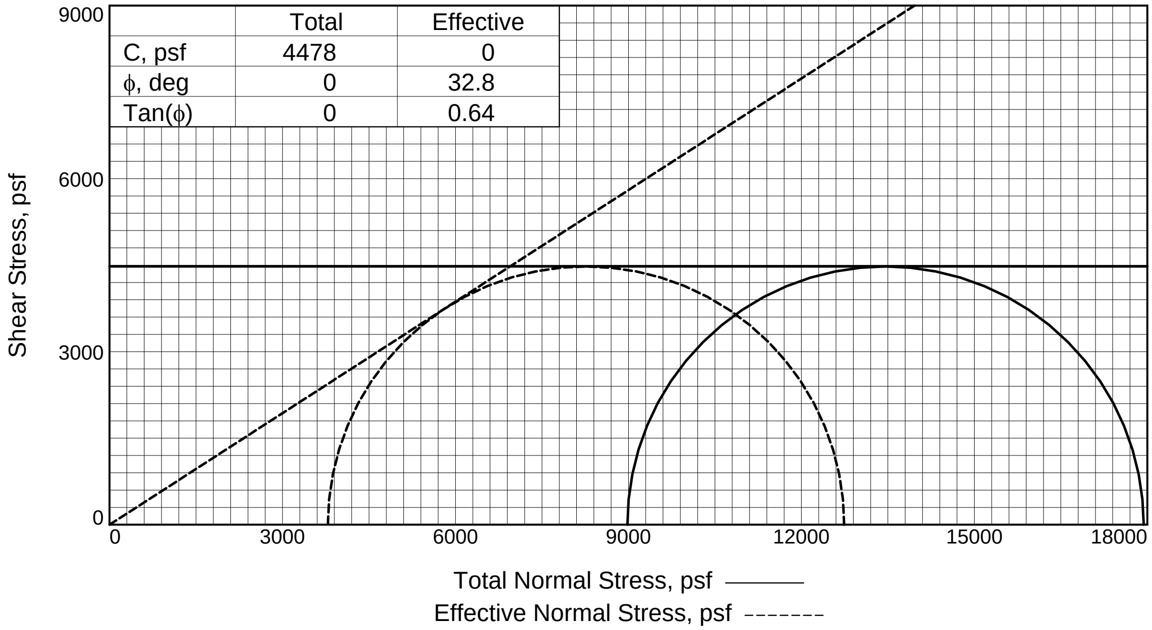
Job Number: 19501-27

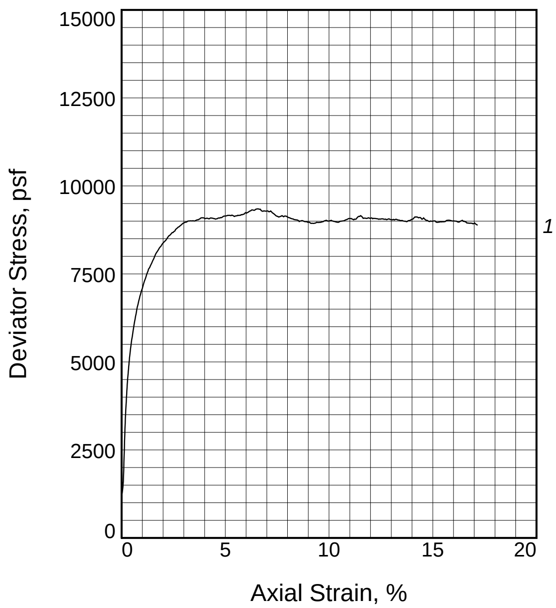
06/21

HARTCROWSER
A division of Haley & Aldrich

Figure
C-9-4

	Total	Effective
C, psf	4478	0
ϕ, deg	0	32.8
Tan(ϕ)	0	0.64





Sample No.		1
Initial	Water Content, %	18.4
	Dry Density, pcf	109.9
	Saturation, %	96.6
	Void Ratio	0.5060
	Diameter, in.	1.98
	Height, in.	4.37
At Test	Water Content, %	15.2
	Dry Density, pcf	116.5
	Saturation, %	96.0
	Void Ratio	0.4204
	Diameter, in.	1.94
	Height, in.	4.27
Strain rate, in./min.		0.004
Eff. Cell Pressure, psi		62.37
Fail. Stress, psf		8955
Excess Pore Pr., psf		5195
Strain, %		3.0
Ult. Stress, psf		8892
Excess Pore Pr., psf		5366
Strain, %		17.1
$\bar{\sigma}_1$ Failure, psf		12742
$\bar{\sigma}_3$ Failure, psf		3786

Type of Test:
CU with Pore Pressures

Sample Type: Thin-walled tube

Description: FAT CLAY (CH)

LL= 34 **PL =** 15 **PI=** 19

Assumed Specific Gravity= 2.65

Remarks: Description and classification based on test results from adjacent SPT sample.

Figure C-10-1

Client: Washington State Department of Transportation

Project: SR542 Squalicum Creek to Bellingham Bay Fish Passage

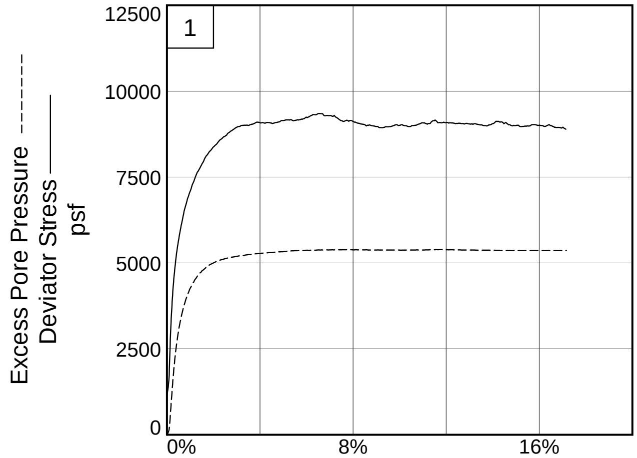
Source of Sample: H-1vw-20 **Depth:** 79.3

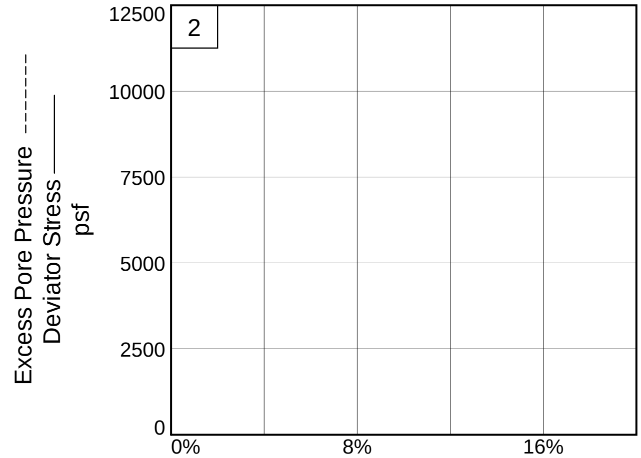
Sample Number: PS-24

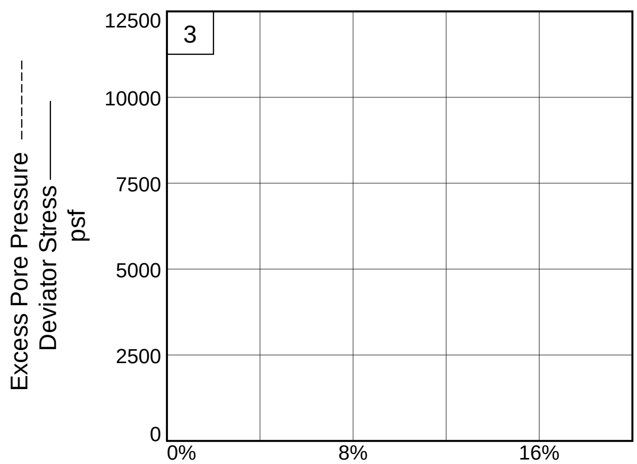
Proj. No.: 19501-27 **Date Sampled:**

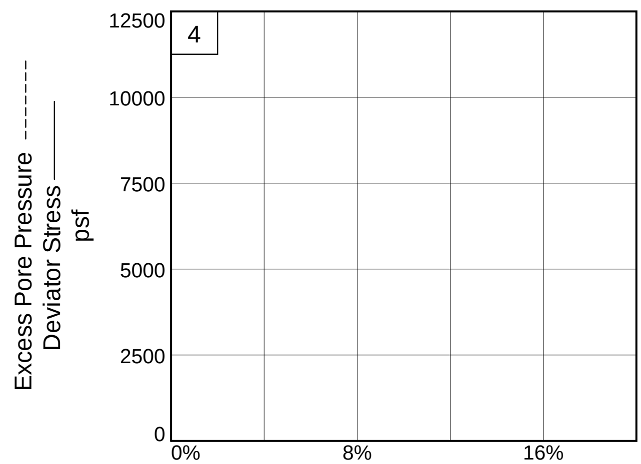
HARTCROWSER
A division of Haley & Aldrich

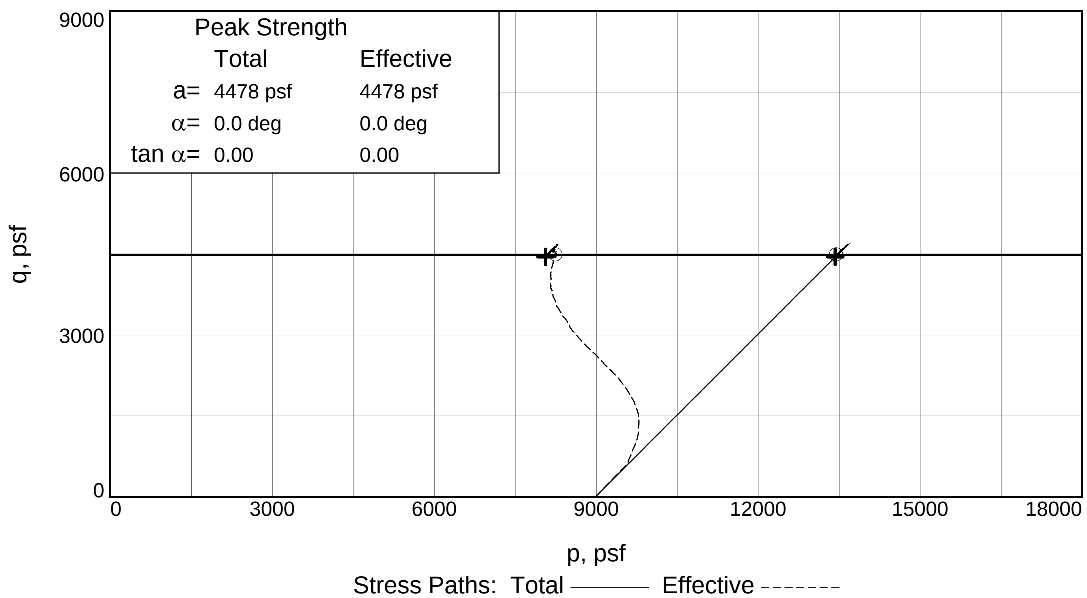
Tested By: PHK











Client: Washington State Department of Transportation

Project: SR542 Squalicum Creek to Bellingham Bay Fish Passage

Source of Sample: H-1vw-20 **Depth:** 79.3 **Sample Number:** PS-24

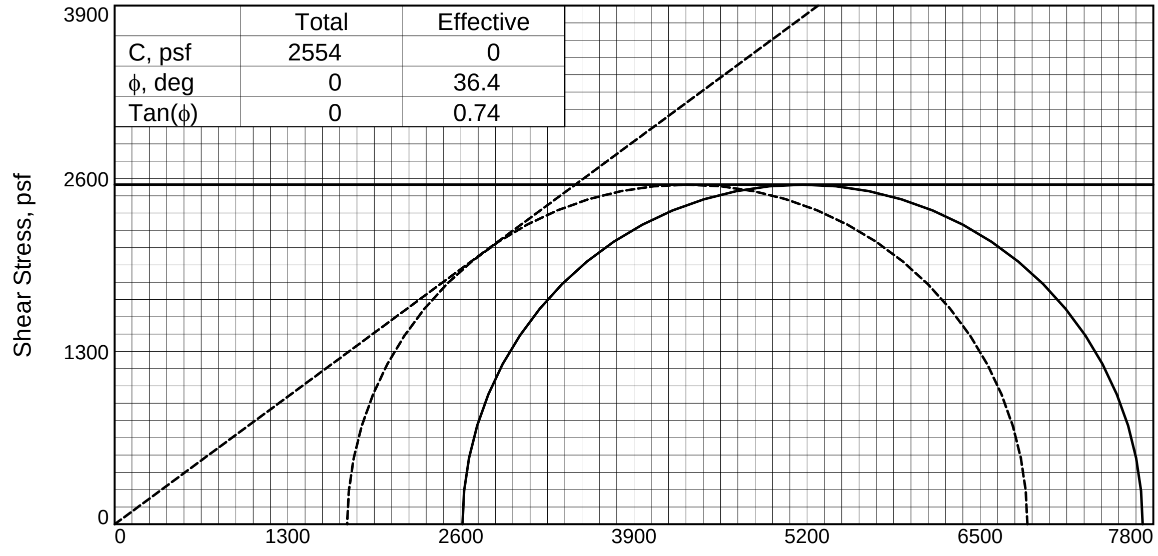
Project No.: 19501-27 **Figure** C-10-2

Hart Crowser, a division of Haley & Aldrich

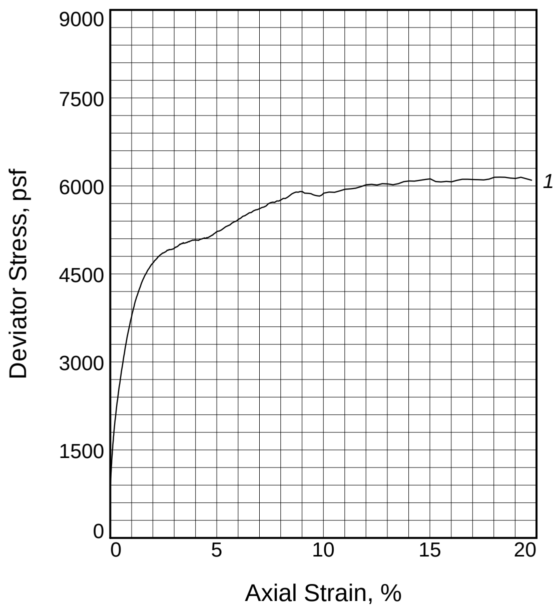
Tested By: PHK

Client: Washington State Department of Transportation
Project: SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Source of Sample: H-1vw-20 **Depth:** 79.3 **Sample Number:** PS-24
Project No: 19501-27 **Figure:** C-10-3

HARTCROWSER
A division of Haley & Aldrich



	Total	Effective
C, psf	2554	0
ϕ, deg	0	36.4
Tan(ϕ)	0	0.74



Sample No.		1
Initial	Water Content, %	15.5
	Dry Density, pcf	116.3
	Saturation, %	97.1
	Void Ratio	0.4226
	Diameter, in.	1.99
	Height, in.	4.96
At Test	Water Content, %	17.1
	Dry Density, pcf	113.3
	Saturation, %	98.7
	Void Ratio	0.4597
	Diameter, in.	2.02
	Height, in.	4.94
Strain rate, %/min.		0.0050
Eff. Cell Pressure, psi		18.14
Fail. Stress, psf		5108
Excess Pore Pr., psf		866
Strain, %		4.4
Ult. Stress, psf		
Excess Pore Pr., psf		
Strain, %		
$\bar{\sigma}_1$ Failure, psf		6854
$\bar{\sigma}_3$ Failure, psf		1746

Type of Test:
CU with Pore Pressures

Sample Type: thin-walled tube

Description: LEAN CLAY (CL)

LL= 26 **PL=** 13 **PI=** 13

Assumed Specific Gravity= 2.65

Remarks:

Client: Washington State Department of Transportation

Project: SR 542 Squalicum Creek to Bellingham Bay Fish Passage

Source of Sample: H-4si-21 **Depth:** 20.4

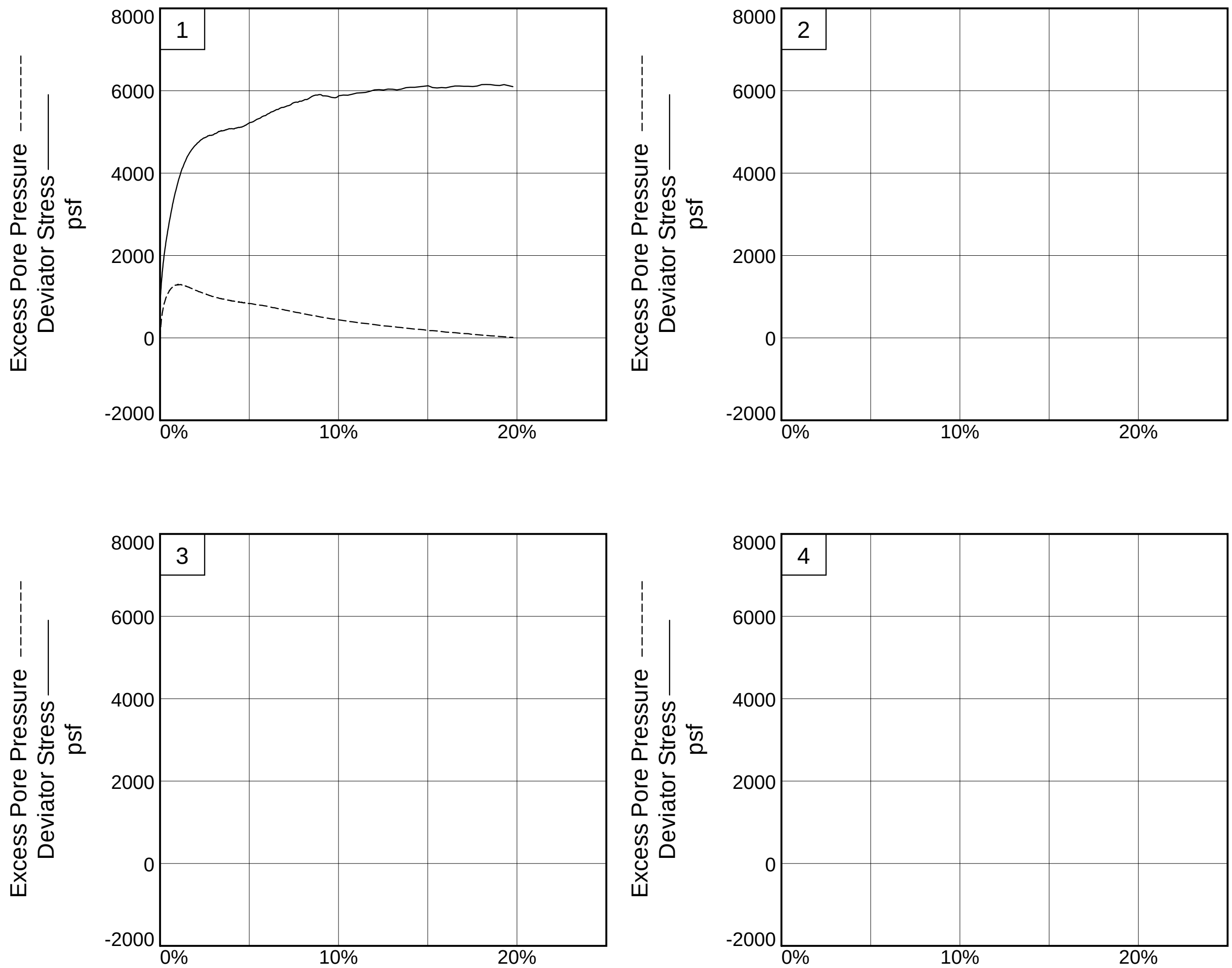
Sample Number: PS-12

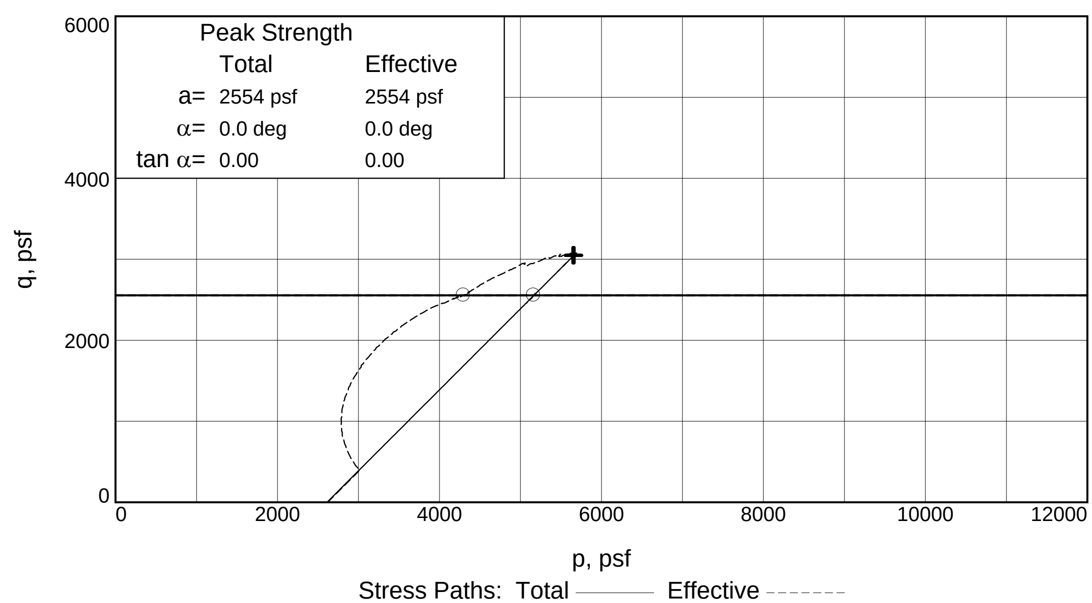
Proj. No.: 19501-27 **Date Sampled:**

HARTCROWSER
A division of Haley & Aldrich

Figure C-11-1

Tested By: PHK **Checked By:** PHK





Client: Washington State Department of Transportation

Project: SR 542 Squalicum Creek to Bellingham Bay Fish Passage

Source of Sample: H-4si-21 **Depth:** 20.4 **Sample Number:** PS-12

Project No.: 19501-27 **Figure** C-11-2

Hart Crowser, a division of Haley & Aldrich

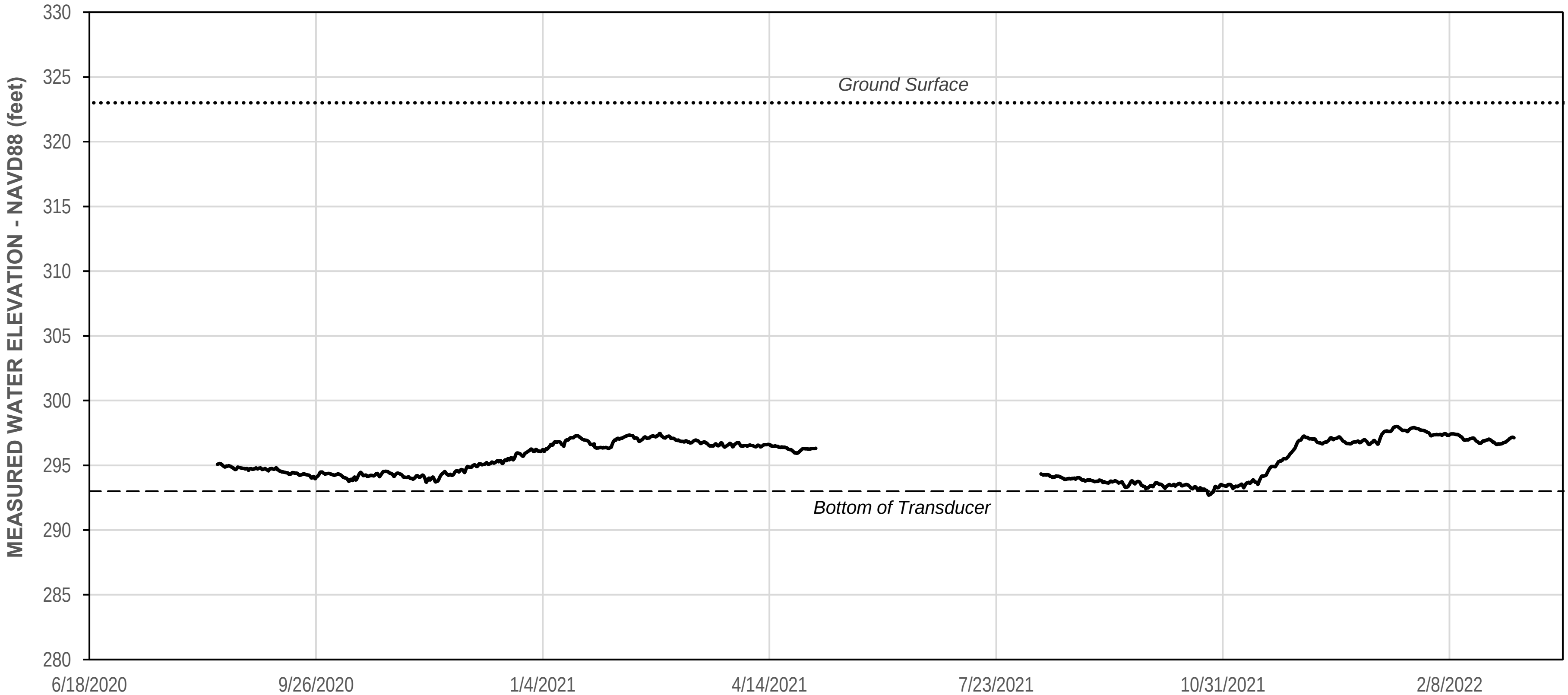
Tested By: PHK **Checked By:** PHK

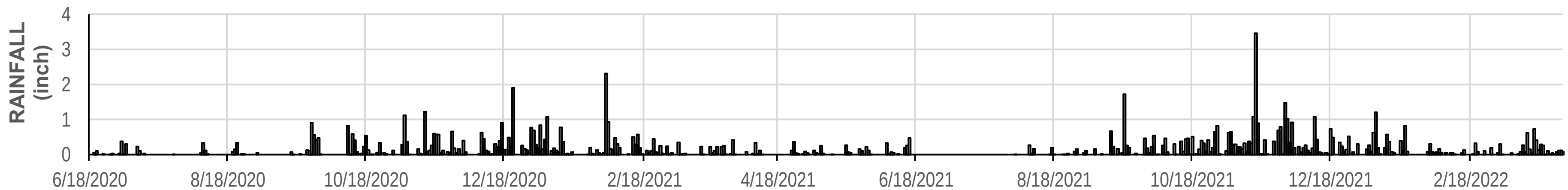
Client: Washington State Department of Transportation
Project: SR 542 Squalicum Creek to Bellingham Bay Fish Passage
Source of Sample: H-4si-21 **Depth:** 20.4 **Sample Number:** PS-12
Project No: 19501-27 **Figure:** C-11-3

HARTCROWSER
A division of Haley & Aldrich

APPENDIX D: GROUNDWATER MONITORING RESULTS

CONTENTS





Exploration Information	
Northing (feet)	660,432
Easting (feet)	1,261,671
Ground Elevation (feet)	323
Total Boring Depth (feet)	130
Date Completed	6/18/2020

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	30 to 35	293 to 288
In-Situ Soil/Rock	Sandy SILT and CLAY	
Highest Reading	25.0	298.0
Lowest Reading	30.3	292.7

** all units in feet*

JOB# XL-6093 STATE ROUTE 542 MILEPOST(S) 03.38 to 03.52

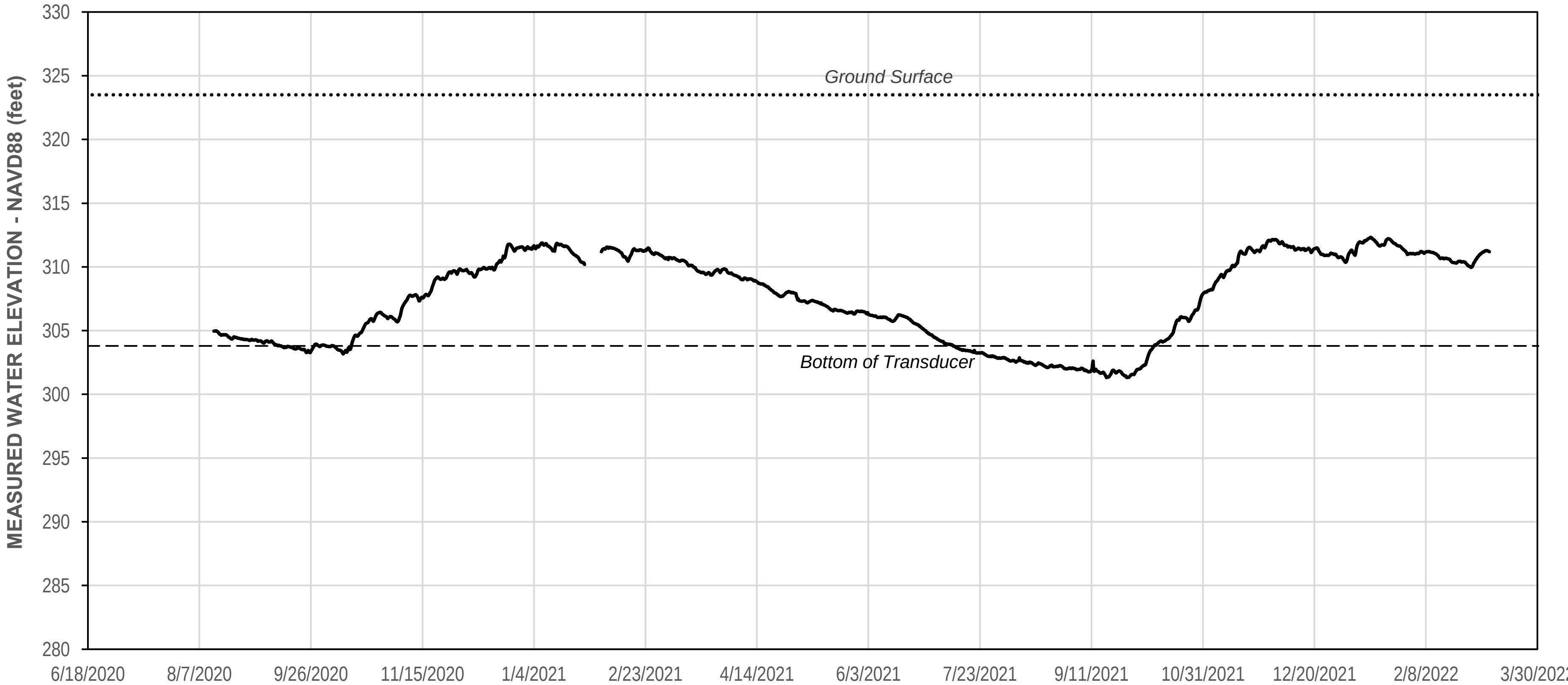
GROUNDWATER MEASUREMENT PLOT
SHALLOW VW BORING H-1VW-20

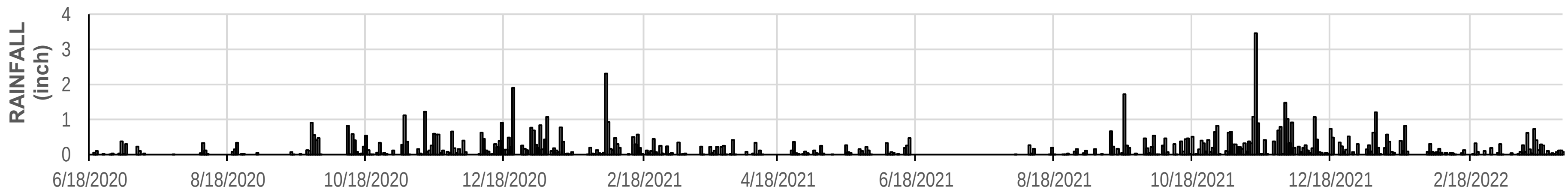
SR-542 SQUALICUM CREEK TO BELLINGHAM BAY

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY M.Liu DATE: 6/21/2022

NOTE:
1. Rainfall data was downloaded from https://www.ncdc.noaa.gov for the BELLINGHAM 2 N, WA US Station (ID US1WAWC0074), located about 3.7 miles southwest of the project site. 2. The data logger was failed and replaced between 5/5/2021 and 8/11/2021





Exploration Information	
Northing (feet)	660,571
Easting (feet)	1,261,835
Ground Elevation (feet)	324
Total Boring Depth (feet)	130
Date Completed	6/17/2020

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	19 to 25	304.5 to 298.5
In-Situ Soil/Rock	Sandy SILT and GRAVEL	
Highest Reading	11.2	312.3
Lowest Reading	22.2	301.3

** all units in feet*

JOB# XL-6093 STATE ROUTE 542 MILEPOST(S) 03.38 to 03.52

GROUNDWATER MEASUREMENT PLOT
SHALLOW VW BORING H-3VW-20

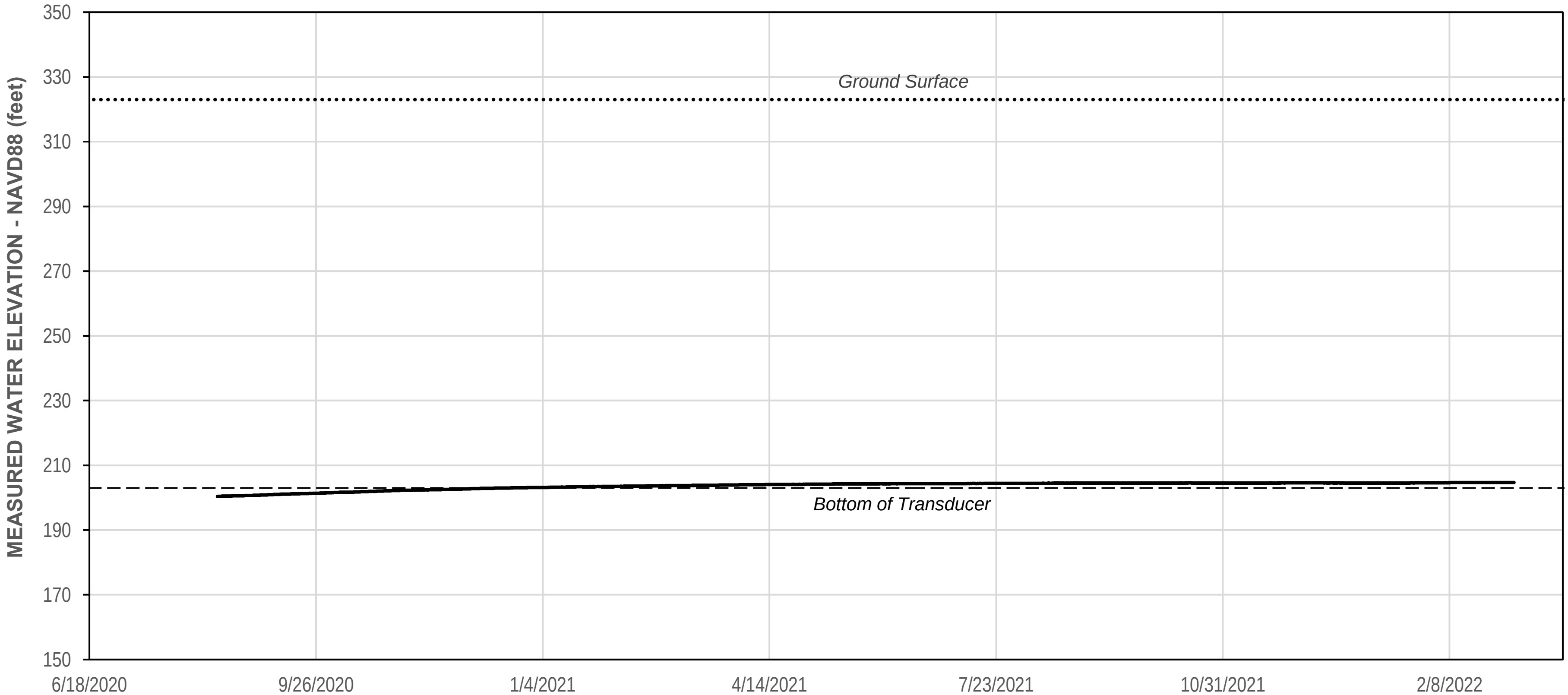
SR-542 SQUALICUM CREEK TO BELLINGHAM BAY

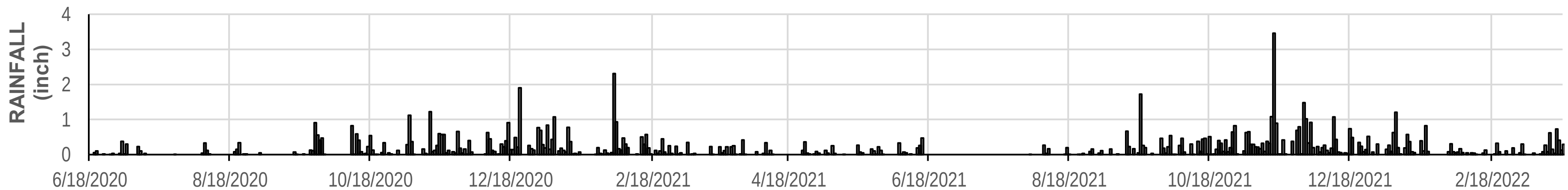
WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY M.Liu DATE: 6/21/2022

NOTE:

1. Rainfall data was downloaded from https://www.ncdc.noaa.gov for the BELLINGHAM 2 N, WA US Station (ID US1WAWC0074), located about 3.7 miles southwest of the project site.





Exploration Information	
Northing (feet)	660,432
Easting (feet)	1,261,671
Ground Elevation (feet)	323
Total Boring Depth (feet)	130
Date Completed	6/18/2020

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	120 to 125	203 to 198
In-Situ Soil/Rock	Sandy SILT and GRAVEL	
Highest Reading	118.3	204.7
Lowest Reading	122.6	200.4

** all units in feet*

JOB# XL-6093 STATE ROUTE 542 MILEPOST(S) 03.38 to 03.52

GROUNDWATER MEASUREMENT PLOT
DEEP VW BORING H-1VW-20

SR-542 SQUALICUM CREEK TO BELLINGHAM BAY

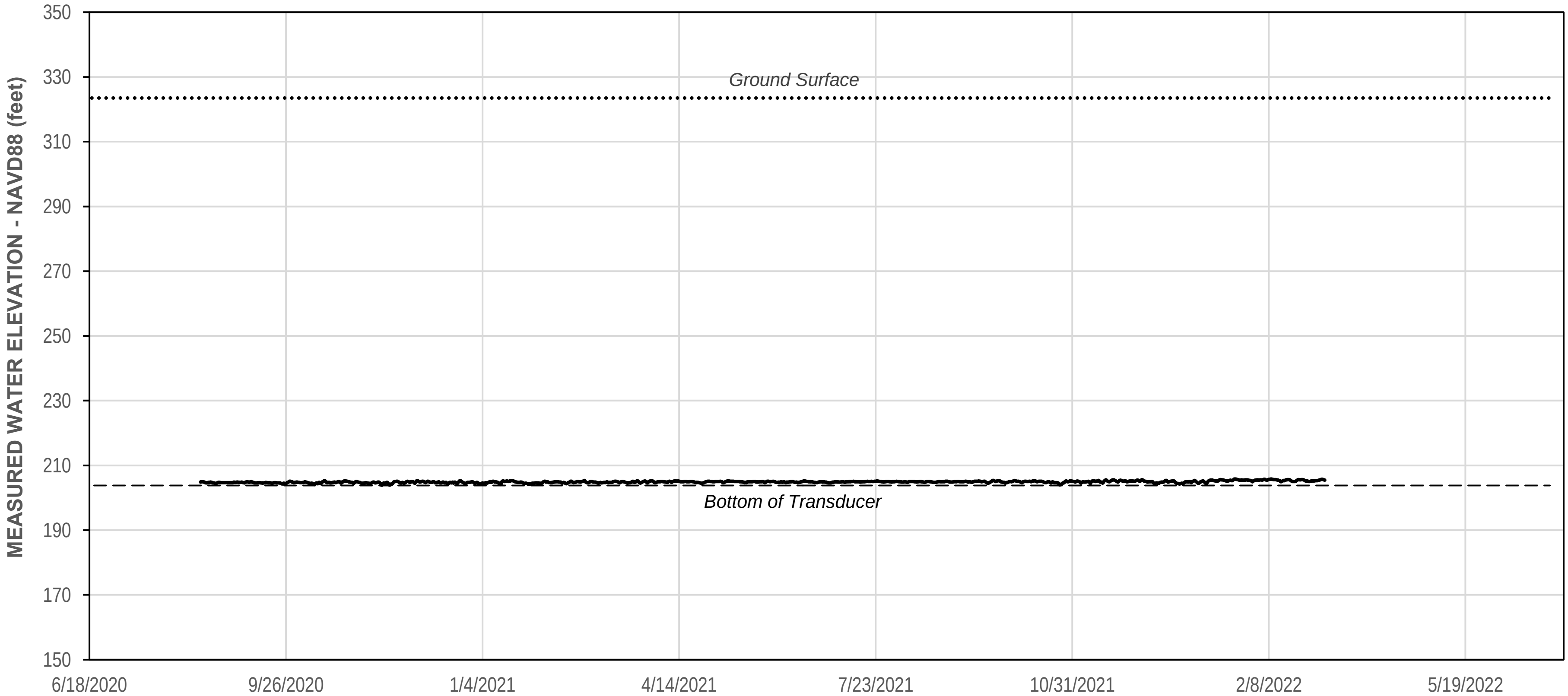
WSDOT

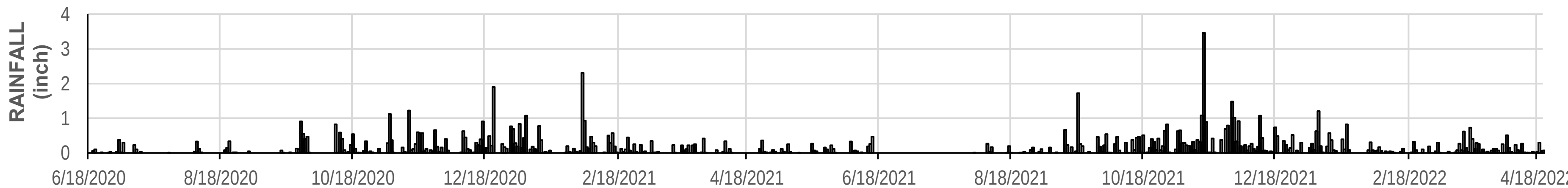
GEOTECHNICAL OFFICE

PREPARED BY M.Liu DATE: 6/21/2022

NOTE:

1. Rainfall data was downloaded from https://www.ncdc.noaa.gov for the BELLINGHAM 2 N, WA US Station (ID US1WAWC0074), located about 3.7 miles southwest of the project site.





Exploration Information	
Northing (feet)	660,571
Easting (feet)	1,261,835
Ground Elevation (feet)	324
Total Boring Depth (feet)	130
Date Completed	6/17/2020

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	119 to 125	204.5 to 198.5
In-Situ Soil/Rock	Sandy SILT and GRAVEL	
Highest Reading	117.7	205.8
Lowest Reading	119.6	203.9

** all units in feet*

JOB# XL-6093 STATE ROUTE 542 MILEPOST(S) 03.38 to 03.52

GROUNDWATER MEASUREMENT PLOT
DEEP VW BORING H-3VW-20

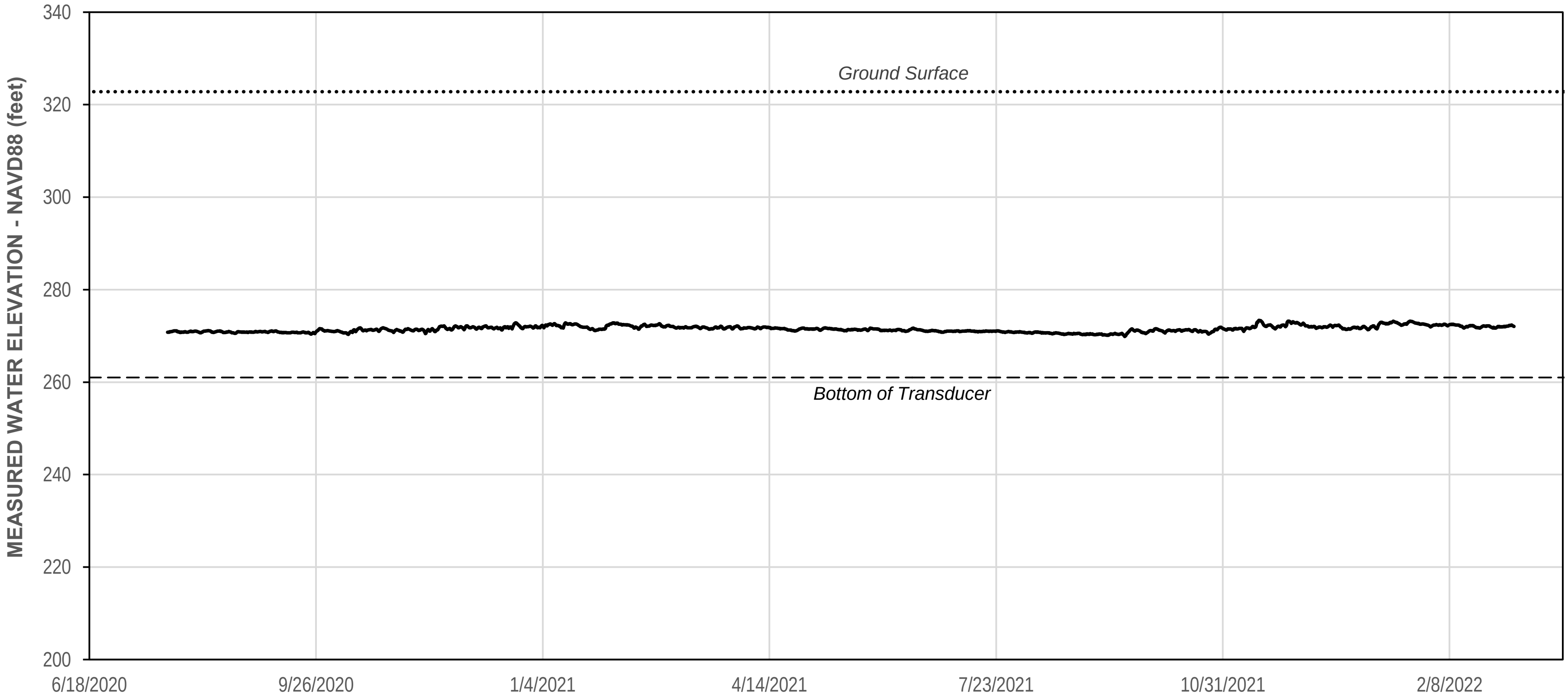
SR-542 SQUALICUM CREEK TO BELLINGHAM BAY

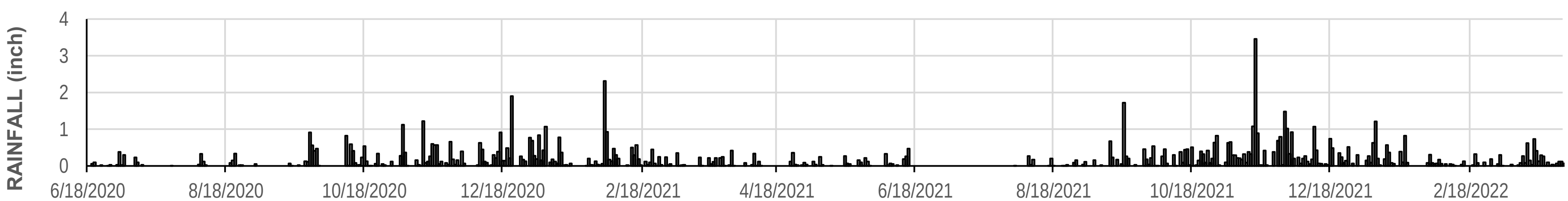
WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY M.Liu DATE: 6/21/2022

NOTE:

1. Rainfall data was downloaded from https://www.ncdc.noaa.gov for the BELLINGHAM 2 N, WA US Station (ID US1WAWC0074), located about 3.7 miles southwest of the project site.





Exploration Information	
Northing (feet)	660,503
Easting (feet)	1,261,763
Ground Elevation (feet)	323
Total Boring Depth (feet)	150
Date Completed	6/23/2020

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	40 to 65	282.8 to 257.8
In-Situ Soil/Rock	Sandy lean CLAY	
Highest Reading	49.4	273.4
Lowest Reading	52.9	269.9

** all units in feet*

NOTE:
Rainfall data was downloaded from https://www.ncdc.noaa.gov for the BELLINGHAM 2 N, WA US Station (ID US1WAWC0074), located about 3.7 miles southwest of the project site.

JOB# XL-6093 STATE ROUTE 542 MILEPOST(S) 03.38 to 03.52

GROUNDWATER MEASUREMENT PLOT
BORING H-2P-20

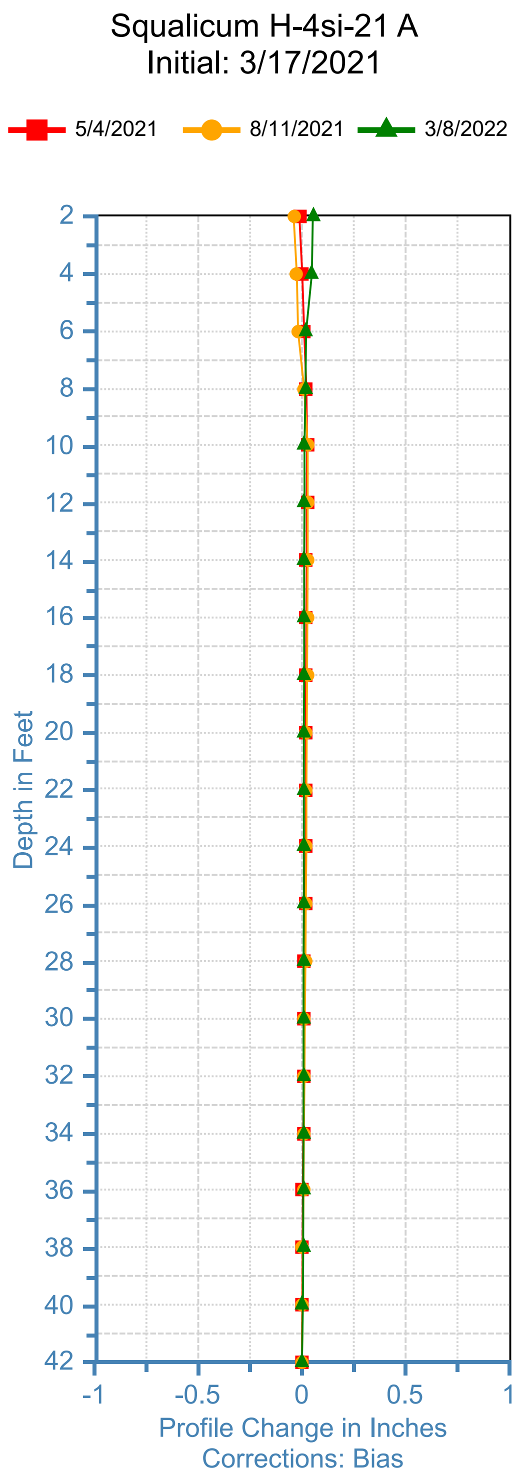
SR-542 SQUALICUM CREEK TO BELLINGHAM BAY

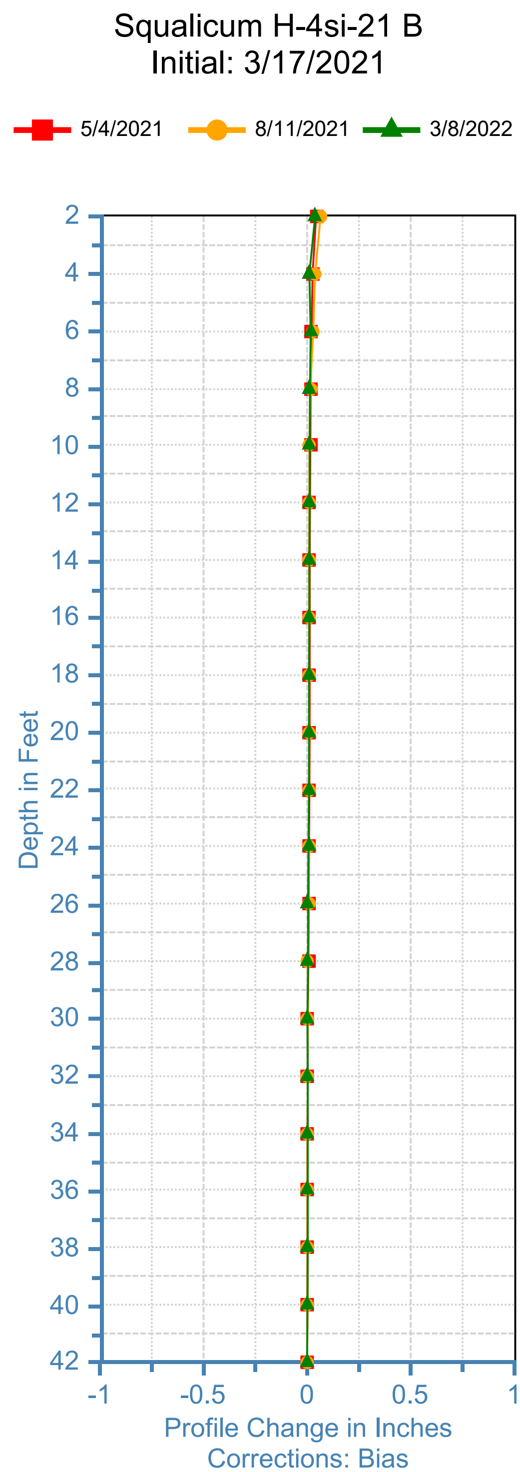
WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY M.Liu DATE: 6/21/2022

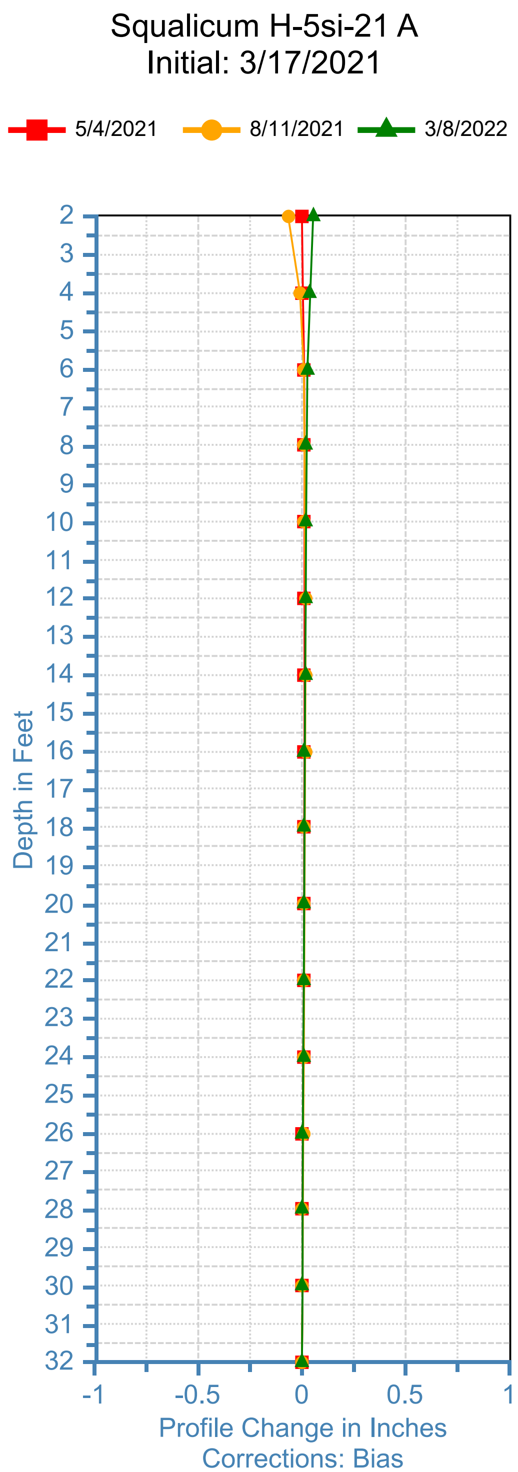
APPENDIX E: INCLINOMETER MONITORING RESULTS

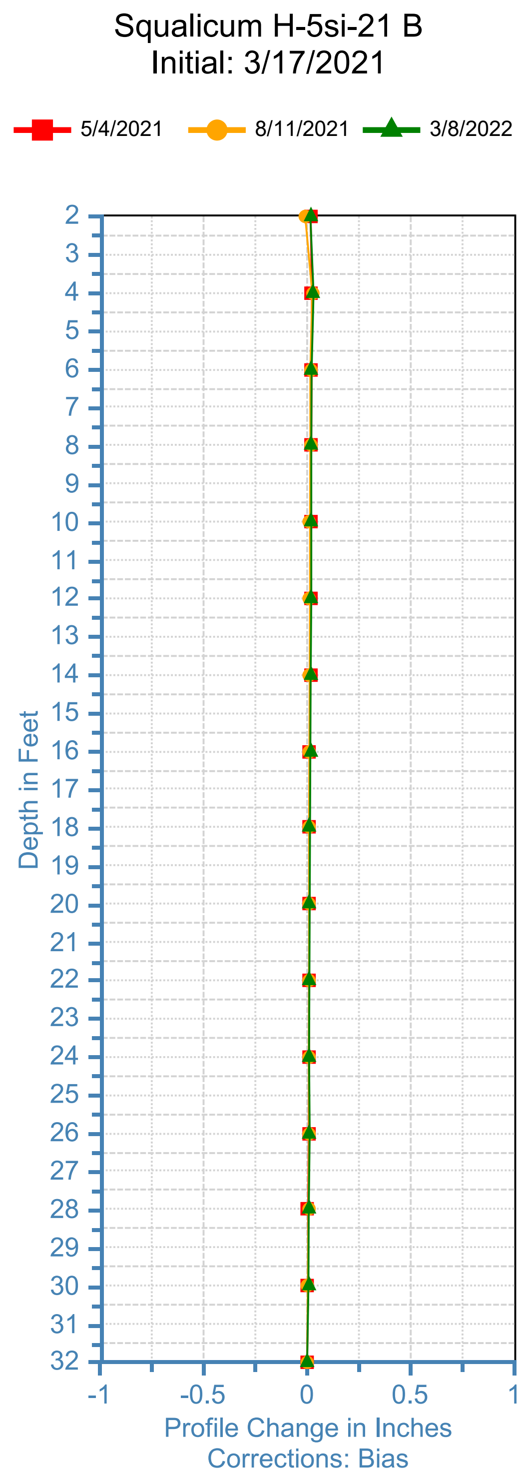
CONTENTS





WSDOT Geotechnical Office Instrumentation Section Olympia, WA	XL6093_SR 542 MP 3.45 Squalicum Cr. to Bellingham Bay H-4si-21 A+ = 357 degrees (true)





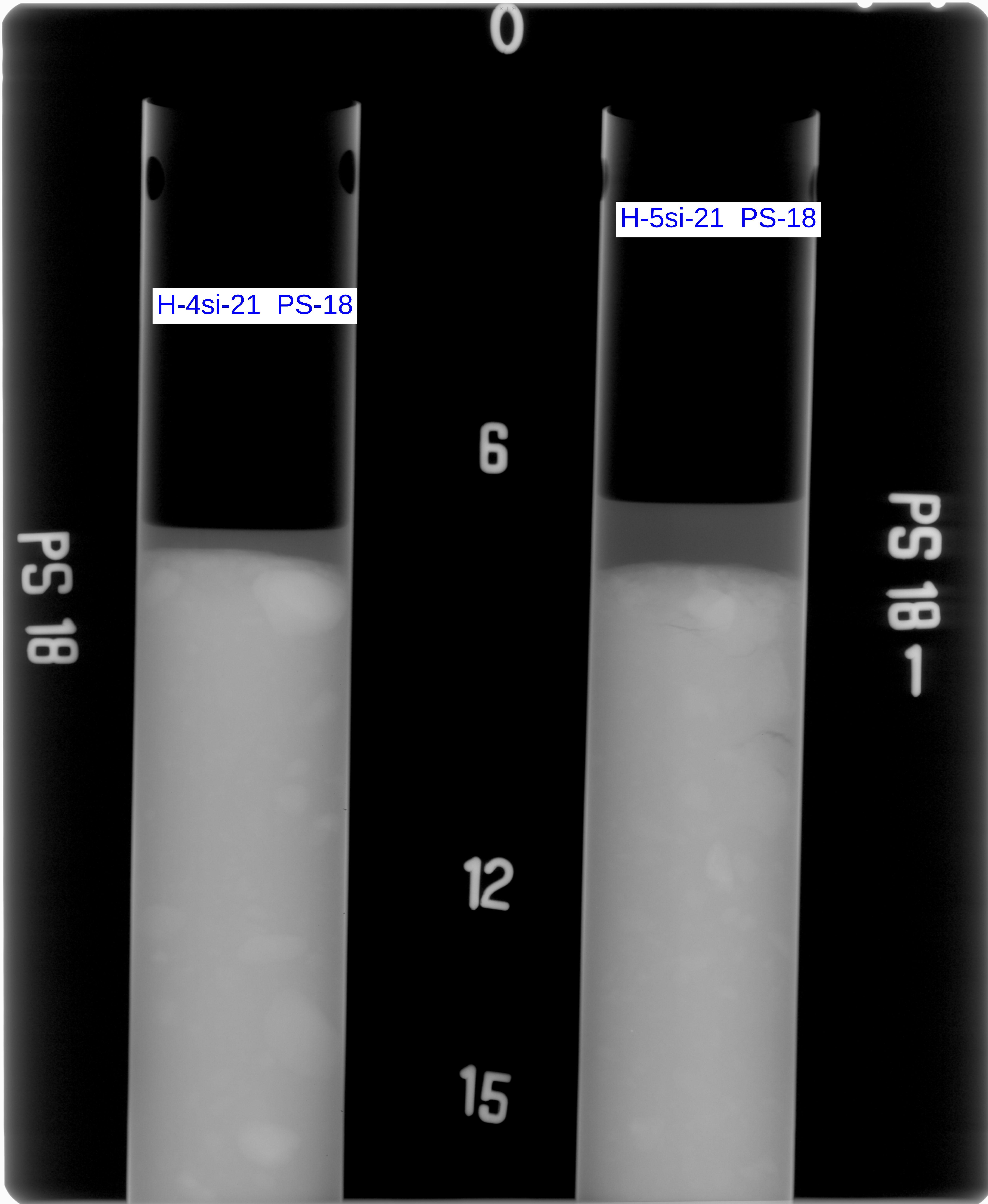
WSDOT Geotechnical Office Instrumentation Section Olympia, WA	XL6093_SR 542 MP 3.45 Squalicum Cr. to Bellingham Bay H-5si-21 A+ = 322 degrees (true)

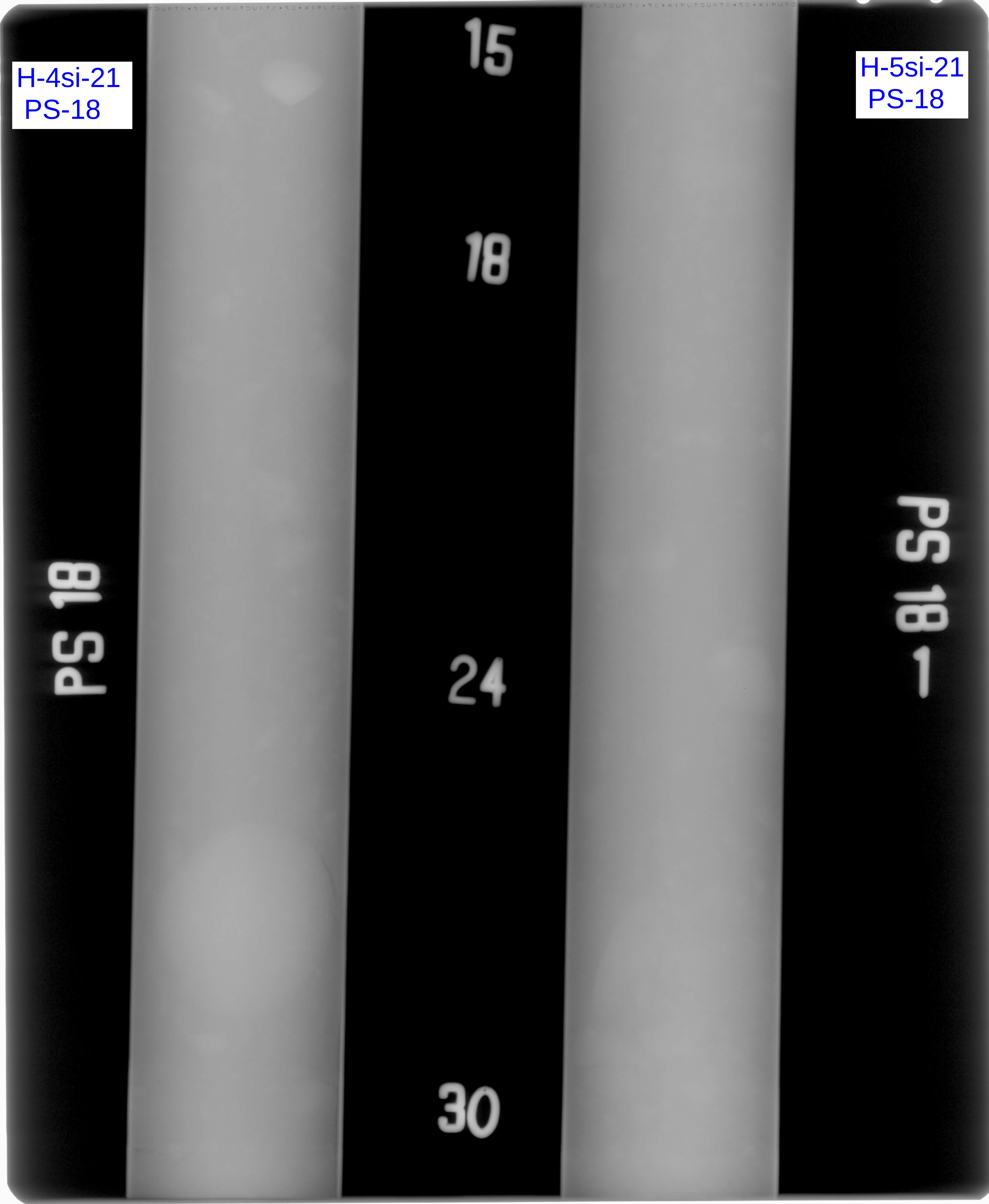
APPENDIX F: X-RAY RESULTS

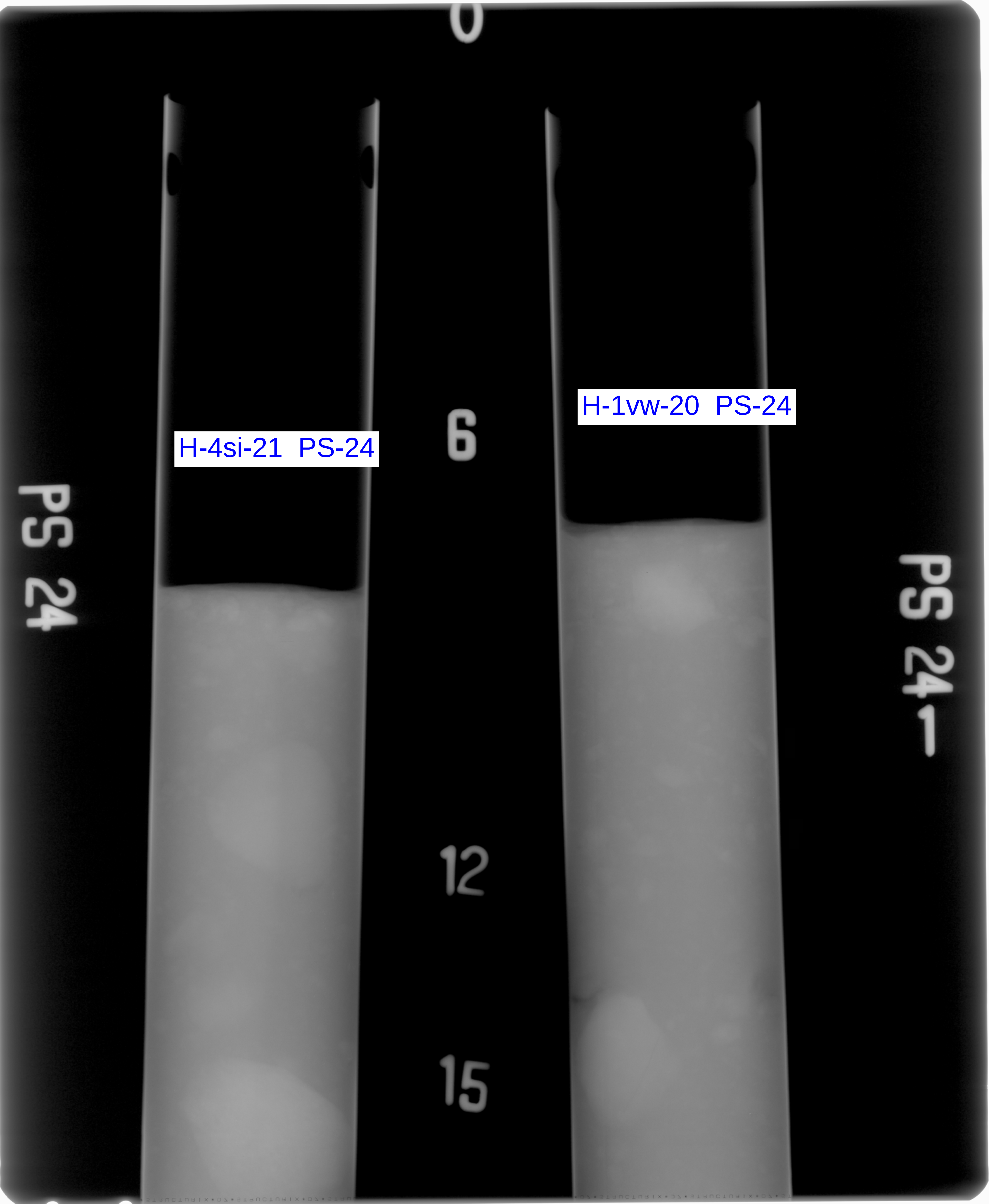
CONTENTS

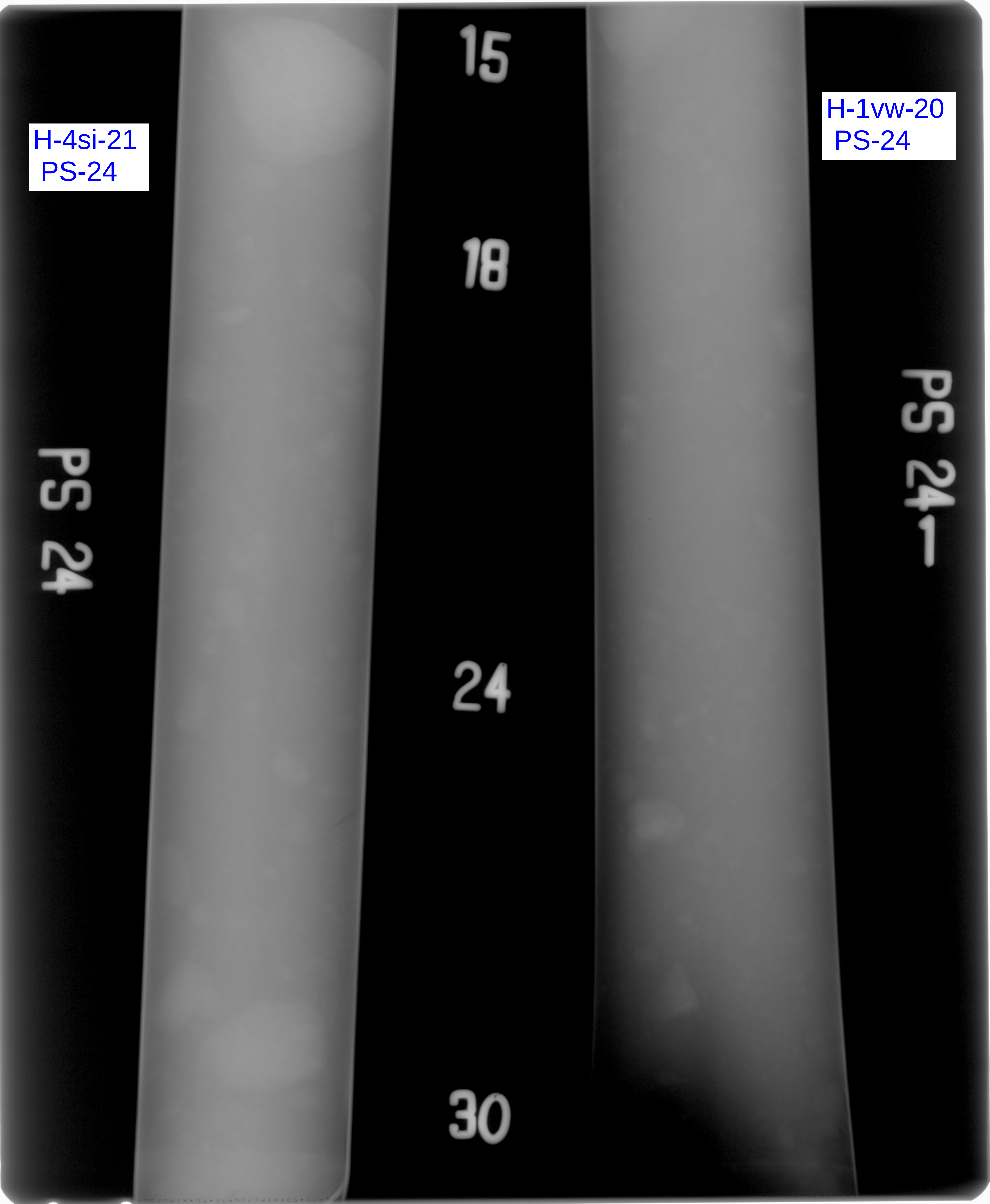
X-ray Results of following Shelby Tube samples from Mistras

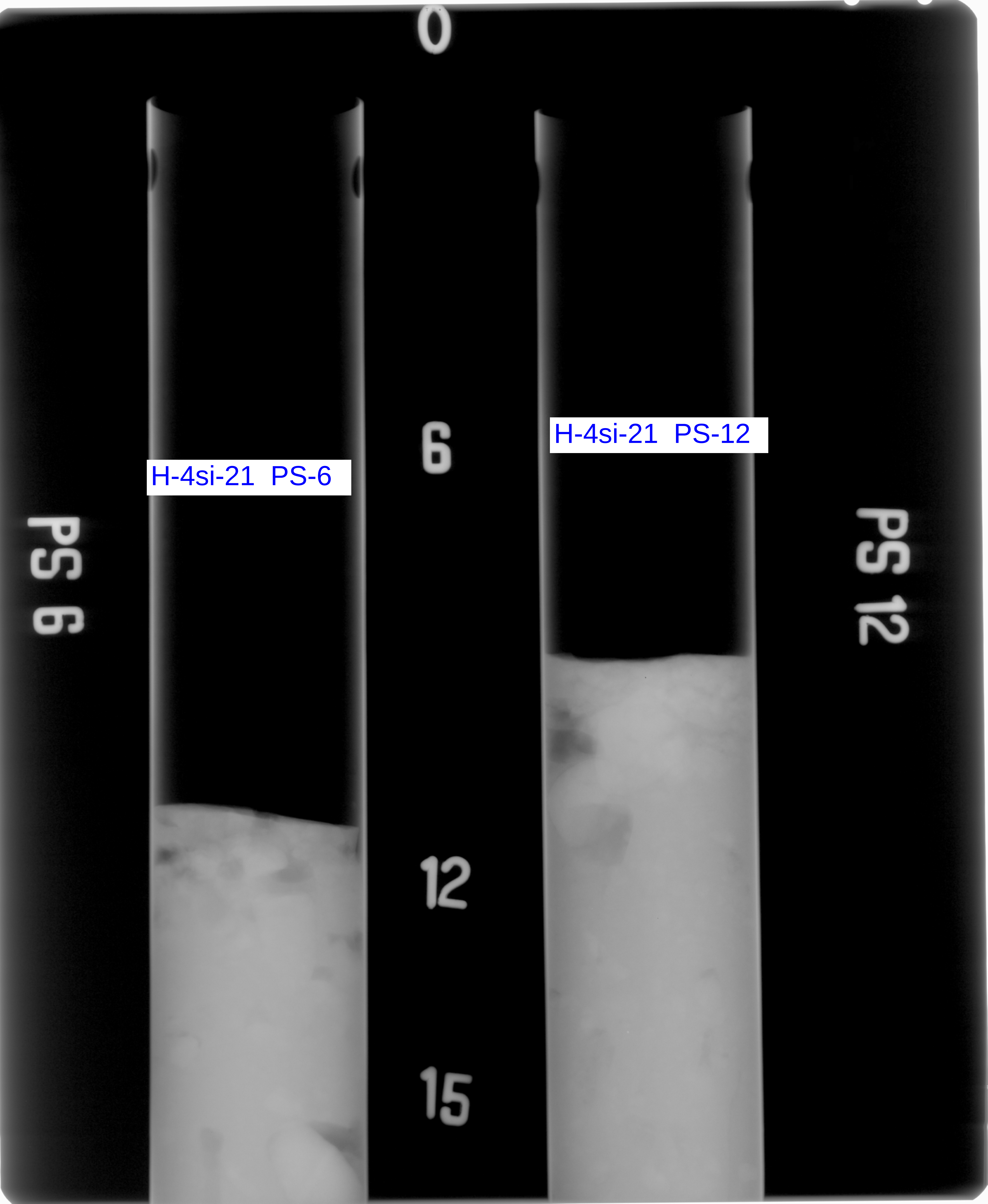
- H-4si-21 PS-18
- H-5si-21 PS-18
- H-4si-21 PS-24
- H-1vw-20 PS-24
- H-4si-21 PS-6
- H-4si-21 PS-12
- H-5si-21 PS-15
- H-5si-21 PS-6
- H-1vw-20 PS-23
- H-2p-20 PS-29
- H-1vw-20 PS-9
- H-2p-20 PS-25

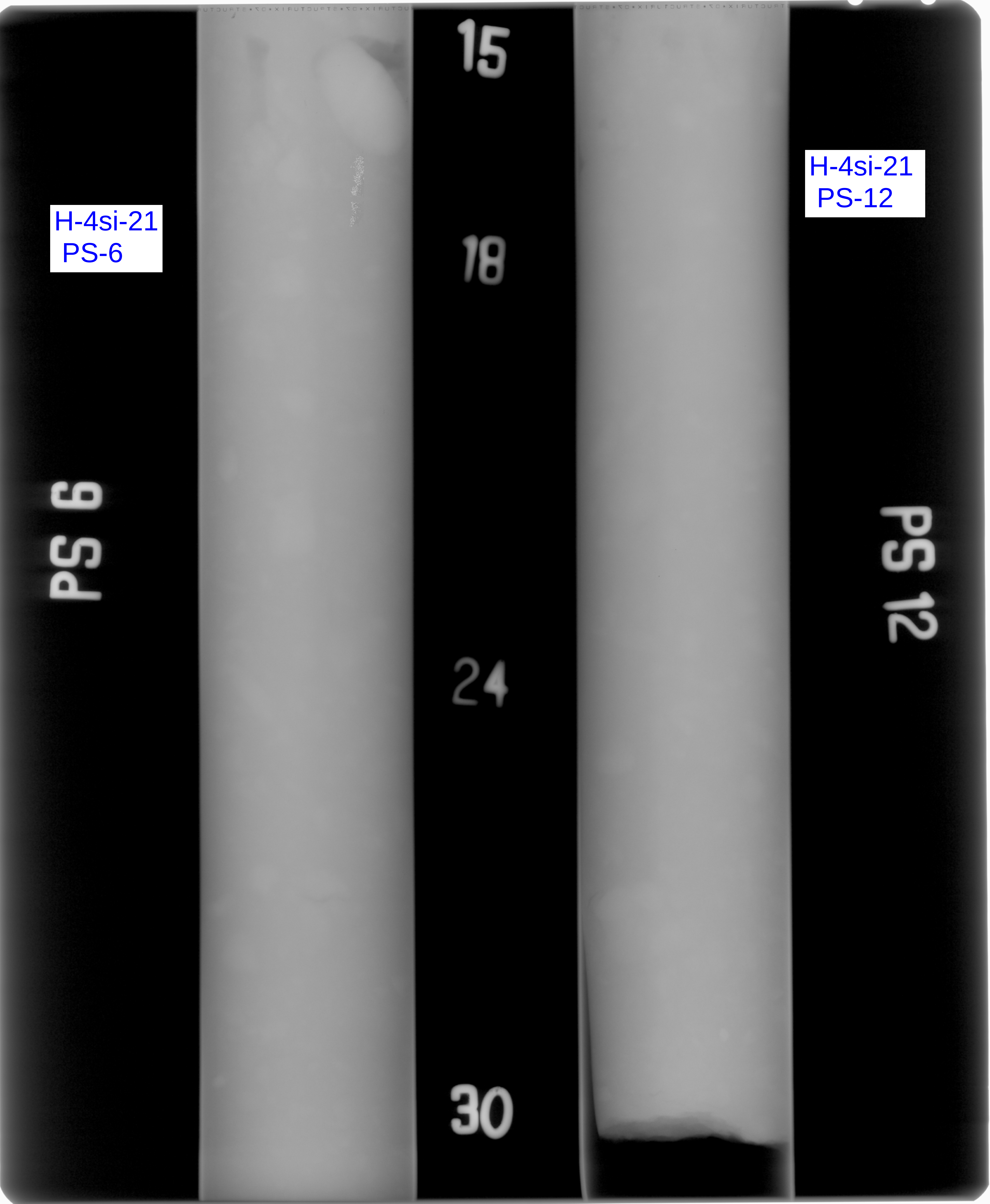

0
H-4si-21 PS-18
H-5si-21 PS-18
6
PS 18
PS 18 - 1
12
15

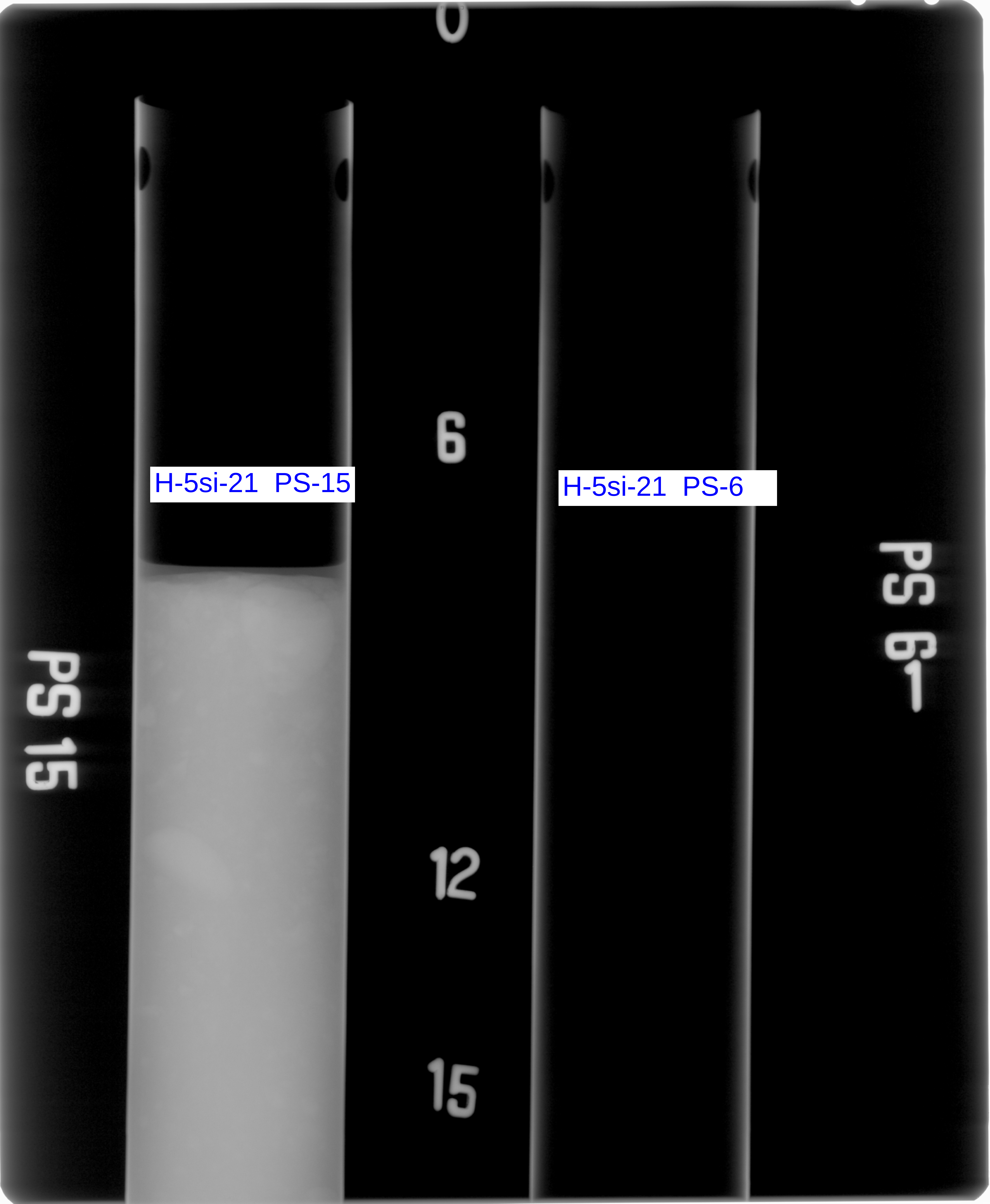

H-4si-21
PS-18
H-5si-21
PS-18
15
18
24
30
PS 18
PS 18 1

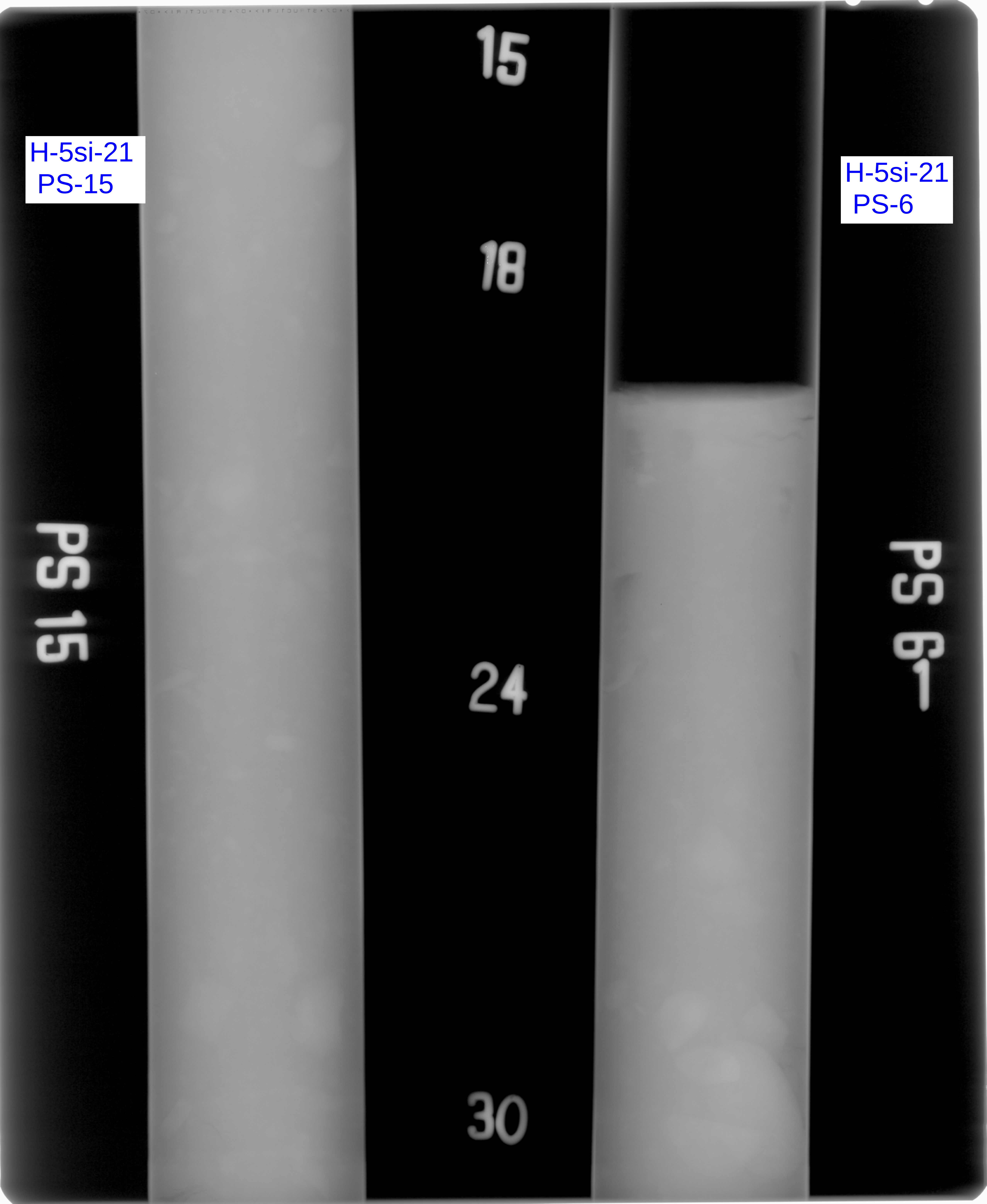

0
H-1vw-20 PS-24
H-4si-21 PS-24
6
PS 24
PS 24 1
12
15

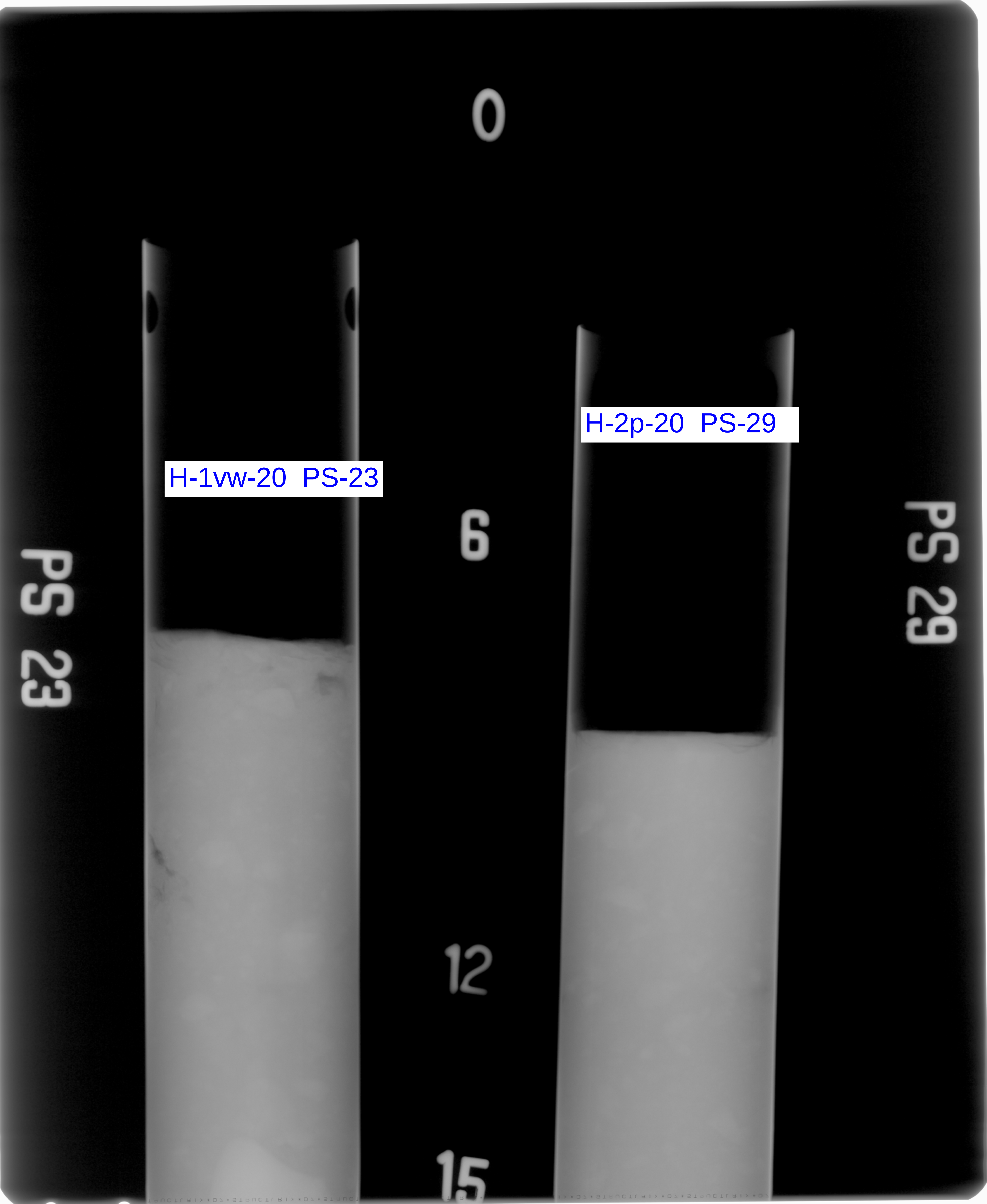

H-4si-21
PS-24
H-1vw-20
PS-24
15
18
24
30
PS 24
PS 24-1

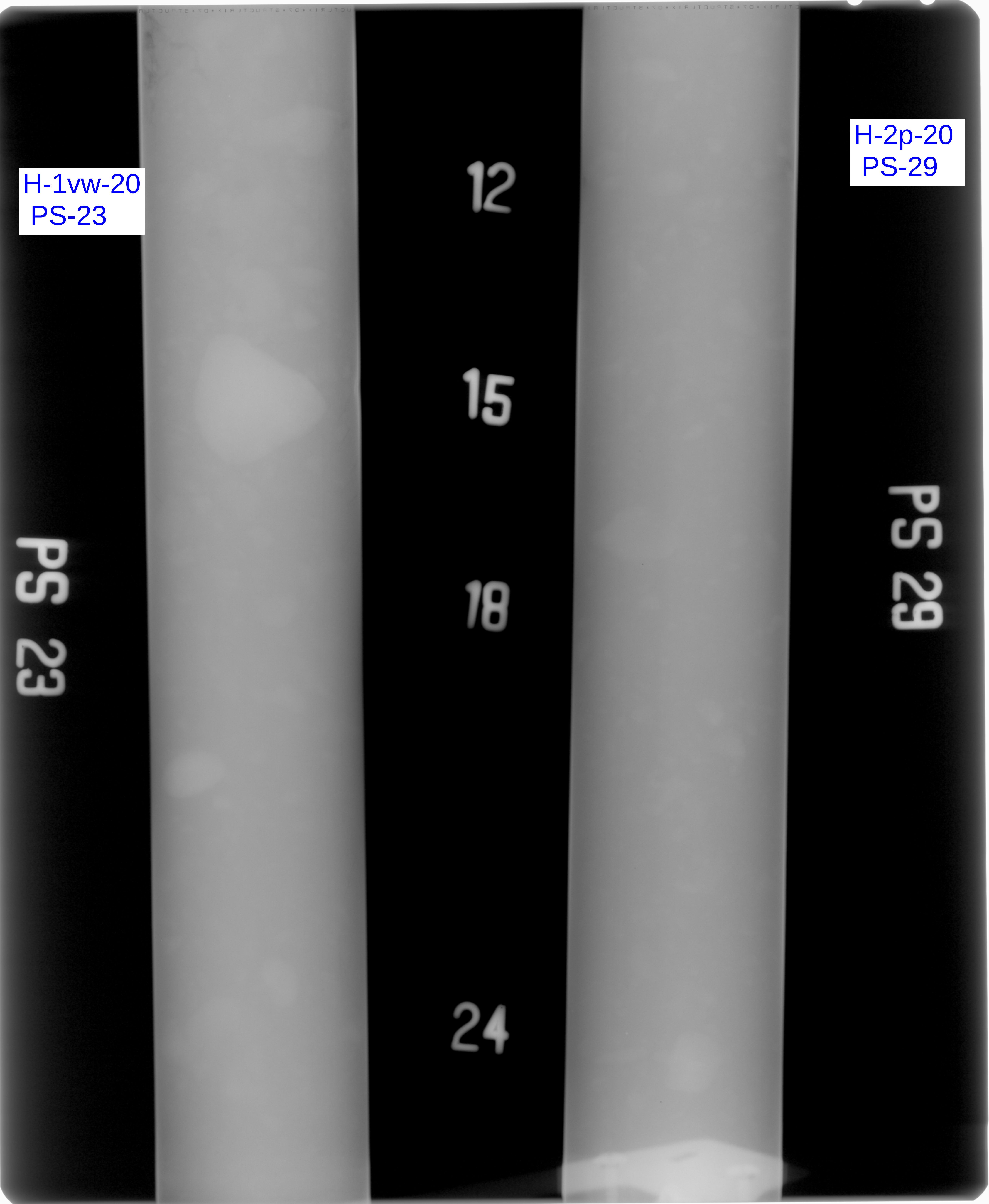

H-4si-21 PS-6
H-4si-21 PS-12
0
6
12
15
PS 6
PS 12

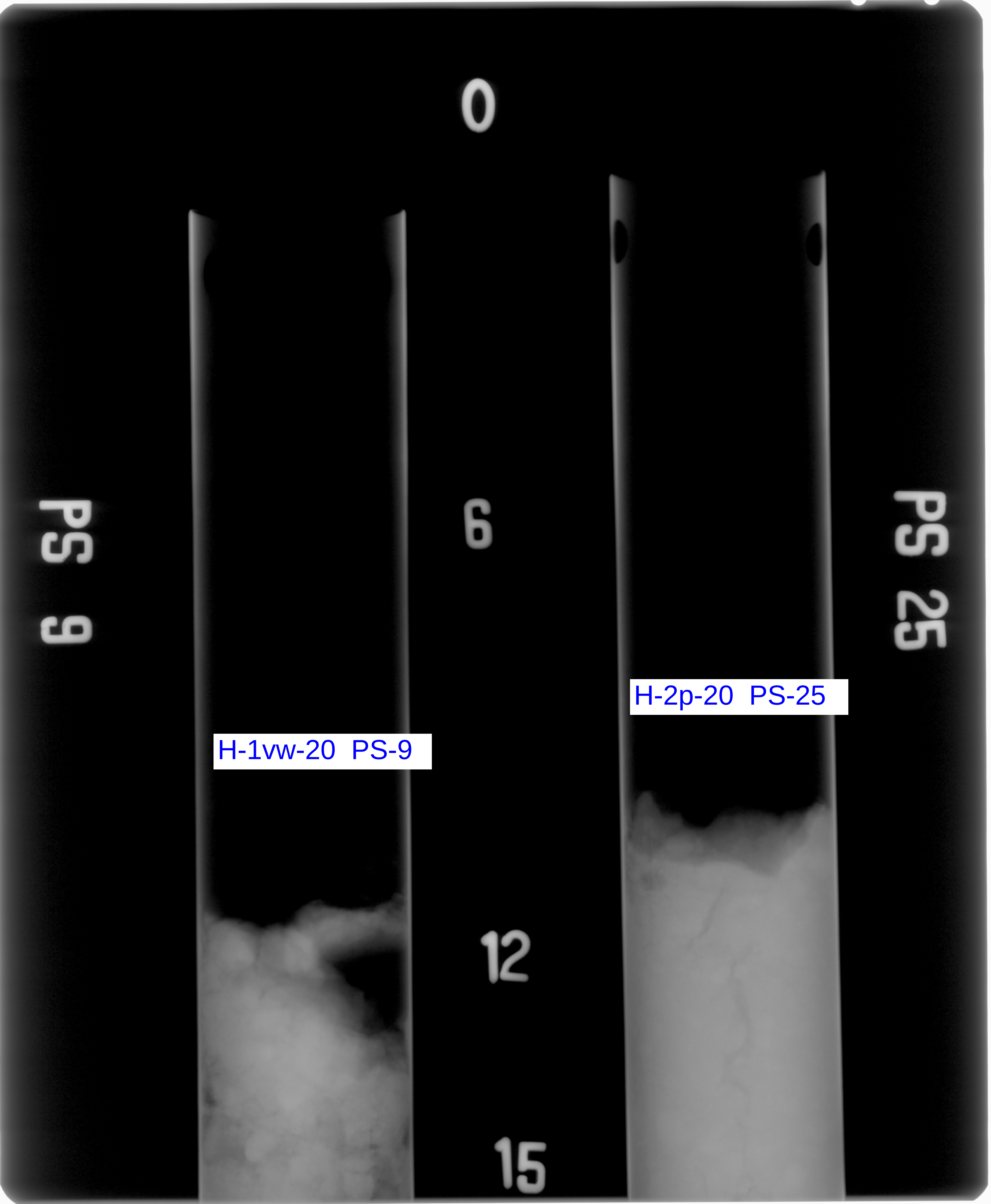

H-4si-21
PS-6
PS 6
15
18
24
30
H-4si-21
PS-12
PS 12

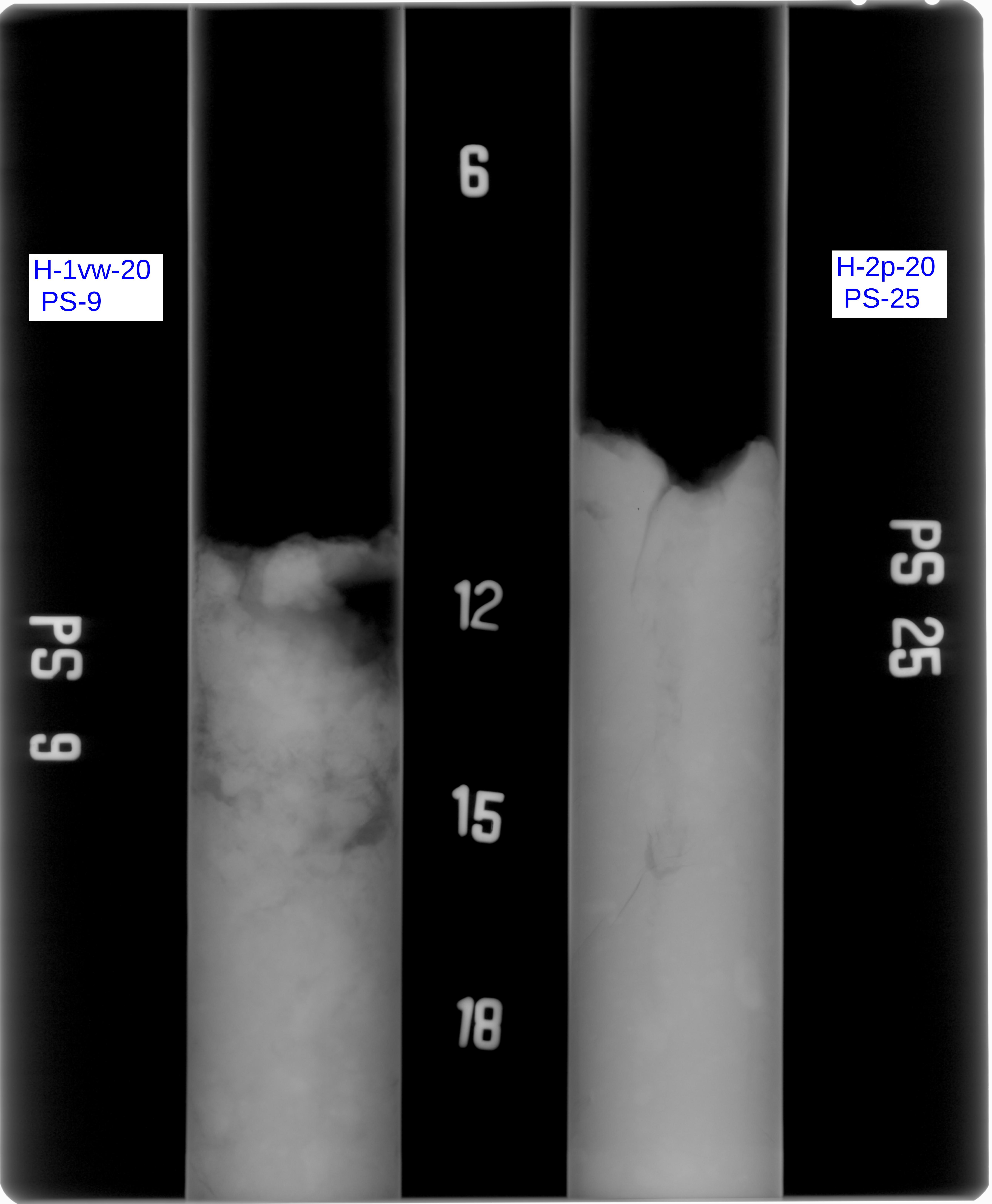

0
6
12
15
H-5si-21 PS-15
H-5si-21 PS-6
PS 15
PS 6


H-5si-21
PS-15
H-5si-21
PS-6
15
18
24
30
PS 15
PS 6-1


0
H-2p-20 PS-29
H-1vw-20 PS-23
6
PS 23
PS 29
12
15


H-1vw-20
PS-23
H-2p-20
PS-29
12
15
18
24
PS 23
PS 29


0
6
12
15
PS 9
PS 25
H-1vw-20 PS-9
H-2p-20 PS-25


H-1vw-20
PS-9
H-2p-20
PS-25
6
12
15
18
PS 9
PS 25